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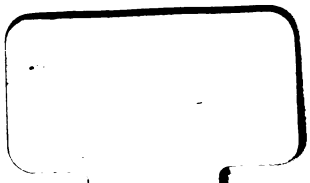

THE

PRACTICAL DESIGNING

OF

STRUCTURAL IRONWORK

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OF

STRUCTURAL IRONWORK

BY

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PREFACE.

MANY of the subjects dealt with in the following chapters will be recognised as having been issued in pamphlet form at intervals during the last ten years for the use of the students in the Engineering Department of the City of London College. Their extended circulation has, however, proved them to be of utility to engineers and architects who are no longer in their pupilage, and they are now rearranged, with several additional notes, in such form as to be handy for reference while designing, so that the volume may with propriety find a place upon every drawing-office table.

The descriptions accompanying the calculations are brief, but will be found ample enough to render the whole of the working clear and definite to any one who has had the ordinary preliminary training in the principles of mechanics and theory of stresses. The opening chapters are expressly made very simple, to remove any possible difficulty that might otherwise be experienced by a beginner in designing, and many subsidiary matters are touched upon throughout the book as indicative of the intimate knowledge of detail which should be shown by a competent draughtsman.

In designing any structure from these notes, the sub-headings should be written on a sheet of foolscap, and the dimensions found by calculation should be entered under

each. This will facilitate the preparation of the design. Some parts, as, for instance, the chapter on Rolled Joists, may be found more useful for explaining the principles than for actual use in designing; but while there is no necessity to work out every rolled joist individually, it must be an advantage to know how to do so when occasion arises.

As a preliminary to actual practice, beginners will find it useful to select an example similar to, but not precisely the same as, each of the examples given, and to work it through upon the same general lines. The course of lectures delivered by the author at the request of the Society of Engineers, entitled 'Strains in Ironwork' (Spon, 5*s.*), is recommended as giving the needful preliminary information upon stresses in a manner easily grasped by non-mathematical men.

LONDON,
60 QUEEN VICTORIA STREET.
September 1894.

[A new series of lectures on Designing Ironwork has been commenced at the City of London College, the notes of which will be published at intervals as opportunity permits. The first part is now ready, the subject being a steel box girder of minimum depth to carry 100 tons over a span of 50 feet. It may be obtained from the author, post free, price 1*s.* 6*d.*, or through any bookseller.]

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THE

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STRUCTURAL IRONWORK.

CHAPTER I.

SMALL CAST-IRON GIRDER.

(See Plate 1.)

DETAILED calculations and notes with design for :—

A cast-iron girder (*a*) supported at the ends (*b*), with parallel flanges (*c*), of uniform section (*d*), to carry a dead load (*e*) of 10 tons uniformly distributed (*f*), over a clear span (*g*) of 10 feet.

The girder will rest upon York stone templates on stock brickwork in mortar.

(*a*) CAST IRON is brittle and crystalline, contains 2 to 6 per cent. of carbon, can only be cast into required shapes, and separate pieces must be joined by bolts, as distinguished from WROUGHT IRON, which is tough and fibrous, contains not more than 0·25 per cent. of carbon, is rolled into bars or plates, and separate pieces are connected mostly by rivets.

(*b*) If supported at one end only it would be called a cantilever, and if it had intermediate supports it would be

called a continuous girder, each involving a different set of calculations. Cast-iron cantilevers or brackets are frequently used, bolted by one end to main structures, or built into and projecting from a wall. Cast-iron girders are usually simply supported at the ends; in ordinary construction they are never made as continuous girders, as they would then require to have flanges of equal section.

(*c*) Large girders, whether of cast or wrought iron, are frequently reduced in depth from the centre to the ends, the top flange forming a parabolic curve, the depth then varying as the stress. Sometimes only the portion over the supporting surface is reduced, the curve being the quadrant of a circle, or nearly so.

(*d*) In cast-iron parallel girders the bottom flange is sometimes reduced in width towards the ends to save metal, leaving an amount proportional to the stress. The general dimensions of each flange are decided by the calculation of the area of the central section, and it is not usual to vary the thickness throughout the length, owing to difficulties in making the pattern. The bottom flange is frequently widened at the ends to increase the bearing surface when resting on a material of less strength. In cast-iron girders the bottom flange is secured to the web, and at the same time stiffened, by angle brackets, webs, or feathers cast on.

(*e*) When the load is fixed and constant it is called dead; a moving or variable load is called a live load and causes greater stress in the girder.

(*f*) A load spread over a girder is called distributed, to distinguish it from a concentrated load which acts at a few points, or at one only. A uniformly distributed load gives one-half the strain upon a girder that an equal load concen-

trated at the centre would give. Distributed loads generally continue over the bearing surfaces, and allowance must be made for this in determining the bearing area. A distributed load is generally more convenient for calculation when given in tons per foot run.

(*g*) The clear span of a girder is the distance between the edges of the bearing surfaces; the effective span will always be greater than this, and may be considered to be the distance from centre to centre of the bearing surfaces.

CALCULATIONS.

Clear span . . . . . 10 feet.

DEPTH OF GIRDER.

Unless otherwise described this is assumed to be the effective or mean depth, or distance between the centres of gravity of the top and bottom flanges, or say centre to centre of the flanges. It should be from $\frac{1}{10}$ to $\frac{1}{15}$ of the span, average say $\frac{1}{12}$. When a parallel girder is built into brickwork it may be necessary to consider the height of the courses in deciding the total or extreme depth of girder.

Span = 10 × 12 = 120 inches, $\frac{1}{10}$ of 120 = 12 inches, $\frac{1}{15}$ of 120 = 8 inches.

Say effective depth = ... 12 inches.

NET LOAD.

10 tons . . . . . 10 tons.

WEIGHT OF GIRDER.

With a uniformly distributed load the weight will approximately equal

$$\frac{\text{Net load tons} \times \text{clear span feet}}{250} = \frac{10 \times 10}{250} = \quad . \quad 0{\cdot}4 \text{ ton.}$$

SAFE LOAD ON BEARING SURFACE.

York or Portland stone . . .	12	tons per ft. sup.
Stock brickwork in mortar, under a carefully bedded template . .	4	„ „

WIDTH OF BEARING SURFACE.

This will be equal to the width of bottom flange, as the girder is to be of uniform section. We must for the present assume a width, but we can arrive at it very nearly by taking the sectional area of bottom flange in sq. inches, as being equal to the distributed load in tons .·. = 10 inches, so that it may be 10×1, $9 \times 1\frac{1}{8}$, or $8 \times 1\frac{1}{4}$; $9 \times 1\frac{1}{8}$ would be a fair proportion, and we may therefore assume the width to be say . . . . . 9 inches.

LENGTH OF BEARING SURFACE.

A certain length will be required to support the load over the clear span, an additional portion of load will be resting immediately above the bearing surfaces, involving a little additional piece of bearing surface, and so on. All this can be provided for in one calculation.

W = net load + weight of girder in tons.
w = load per foot run in tons.
s = safe load on bearing surface in tons per sq. foot.
b = breadth of bearing surface in feet.
l = length of „ „ „

The load on bearing surface consists of . . $W + w l$
and resisting it we have . . . . . $s b l$
and they must be equal, therefore $W + w l = s b l$,
or, put into figures,

$$(10 + \cdot 4) + \frac{10 \cdot 4}{10} l = 12 \times \frac{9}{12} l,$$

which is the same as $10 \cdot 4 + 1 \cdot 04 l = 9 l$,
or by transposition $9 l - 1 \cdot 04 l = 10 \cdot 4$,
or $7 \cdot 96 l = 10 \cdot 4$ $\therefore$ $l = \frac{10 \cdot 4}{7 \cdot 96} = 1 \cdot 3$ feet,
half of which would be at each end, $1 \cdot 3 \times 12 \div 2 = 7 \cdot 8$,
say . . . . . 8 inches.

But, as a rule, if a bearing surface is not more than 1 foot wide, the length should not be less than the width;
$\therefore$ say . . . . . 9 inches.

In small girders the bearing surface will usually be sufficient if the length of girder resting at each end is equal to the width of bottom flange, and a margin of 3 to 4½ inches is allowed at the sides and end for the size of stone to spread the weight over a sufficient surface of brickwork.*

* When the bearing surface has to be taken on a wall standing in the same direction as the girder, and not thicker than the width of bottom flange, the stone will be flush with the brickwork, and the safe load per square foot on brickwork (4 tons) must be taken instead of safe load on stone (12 tons). This will make a considerable length of bearing surface, and will moreover so fix the ends as to put the top flange of the girder, over edge of bearing surface, in tension. In such a case it will not be safe to make the top flange smaller than half the area of bottom flange.

TOTAL LENGTH OF GIRDER.

Clear span + two bearing surfaces = 10 + 1·5 = 11·5 feet.

EFFECTIVE SPAN.

Owing to the camber given to a girder, it may happen that it is supported only at the extreme ends, and, in the absence of sufficient camber, it might rest only on the inside edges of the bearing surfaces; hence it will be proper to take as the effective span the distance from centre to centre of the bearing surfaces.

Clear span 10 feet + one bearing surface 9 inches

= 10·75 feet.

(Taking one bearing surface complete is of course the same as taking two halves.)

EFFECTIVE LOAD.

$$\text{Load per foot run} = \frac{\text{net load tons}}{\text{clear span feet}} = \frac{10}{10} = 1 \text{ ton.}$$

Effective load = (Effective span feet × load tons per foot run) + weight of girder in tons = (10·75 × 1) + ·4

= 11·15 tons.

MAXIMUM STRESS IN FLANGES.

With a uniformly distributed load the maximum stress will be in the centre of span, and in all cases is equal in both flanges at the same vertical section.

$$= \frac{\text{Effective load tons} \times \text{effective span feet}}{\text{Coefficient of reaction} \times \text{effective depth feet}} = \frac{W\,l}{k\,d}$$

$$= \frac{11{\cdot}15 \times 10{\cdot}75}{8 \times 1} = 14{\cdot}983, \text{ say} \quad . \quad . \quad . \quad 15 \text{ tons.}$$

COEFFICIENT OF REACTION.

The coefficient of reaction (k) depends upon the position of the load, and mode of supporting girder, as explained in the author's 'Strains in Ironwork,' pp. 24, 25.

When fixed one end, loaded the other,	$k = 1$
„ „ load distributed,	$k = 2$
Supported both ends, load in centre	$k = 4$
„ „ load distributed	$k = 8$
Fixed both ends, load in centre	$k = 6$ or 8*
„ „ load distributed	$k = 12$ or 16*

PARABOLA OF STRESS.]

If a parabola be drawn to any scale with a base equal to the effective span, and a height equal to the maximum longitudinal stress, an ordinate from the base at any point will give the stress at that point. To draw a parabola of base A B and height C D (Fig. 1), divide A C into any number of equal parts (preferably by successive bisections) and draw perpendiculars 1, 2, 3, &c.; divide A E into the same number of equal parts 1, 2, 3, &c.; put a needle at D, and with edge of scale against D and 1, 2, 3, respectively on A E, draw intersections across 1, 2, 3, &c., on A C; these intersections will be points in the parabola, and for ordinary cases they may be joined by straight lines without differing much from the true curve.

If the height of parabola is made equal to the effective

* Authorities differ, but the lower values are more usual.

depth of girder in centre, then the height at intermediate parts shows the necessary depth of girder at those parts, with flanges of uniform sectional area. In this case a parallel girder is specified, so that an unavoidable waste of material will occur, viz. the amount of web between the parabola and top flange.

SECTIONAL AREA OF BOTTOM FLANGE.

The safe dead load on cast iron in tension is $1\frac{1}{2}$ tons per sq. inch.

$$\therefore \frac{15 \text{ tons}}{1{\cdot}5} = \quad , \quad . \quad . \quad , \quad . \quad 10 \text{ sq. inches.}$$

This is more direct than multiplying the load by a factor of safety, and then estimating the section from an assumed breaking weight. In the absence of a very stringent specification, followed by competent inspection during manufacture, and actual testing of the material, it is impossible to say what the breaking weight would be, whereas in limiting the working stress to $1\frac{1}{2}$ tons per sq. inch, we know that we are on the safe side, and that it would require from 4 to 6 times the working load to cause fracture under ordinary circumstances.

HODGKINSON'S FORMULA.

A common formula for net breaking weight in tons in centre of cast-iron girder, founded on Hodgkinson's experiments, is :—

W = breaking weight in tons in centre.
a = area of tension flange in sq. inches.

d = total depth in inches.
l = clear span in inches.

$$W = \frac{26\,a\,d}{l}.$$

Applying it to the present case

$$\frac{26 \times 10 \times \text{say } 13}{120} = 28{\cdot}17,$$

which for a distributed load would be equivalent to $28{\cdot}17 \times 2 = 56{\cdot}34$; and allowing a factor of safety of $\frac{1}{5}$, $\frac{56{\cdot}34}{5} = 11{\cdot}27$ tons safe working load.

FACTOR OF SAFETY

is an amount fixed by practical experience, varying with the material used, and the manner of using. It is the ratio of the greatest safe stress to the ultimate resistance of the material, such as $\frac{1}{4}$, $\frac{1}{10}$, &c., and the calculated resistance of any section multiplied by the factor of safety suitable to the circumstances, will give the safe working load.

If structures never deteriorated they might be loaded to $\frac{1}{3}$ of their breaking weight with perfect safety, but to guard against ordinary contingencies $\frac{1}{4}$ of the breaking weight is the maximum permanent load allowable under any circumstances, although cast iron is usually limited to $\frac{1}{5}$ or $\frac{1}{6}$.

SIZE OF BOTTOM FLANGE.

The width of bottom flange should be generally from $\frac{3}{5}$ to $\frac{4}{5}$ the mean depth of girder, and the thickness from $\frac{1}{6}$ to $\frac{1}{10}$

the width, but never less than $\frac{1}{12}$ the width, as there is great risk of damage to a thin flange by careless handling.

$\frac{3}{5}$ of 12 = 7·2, $\frac{4}{5}$ of 12 = 9·6.

Width say . . . . . 9 inches.

$\frac{1}{6}$ of 9 = 1·5, $\frac{1}{10}$ of 9 = ·9,

but the area must be 10 sq. inches. $\therefore \frac{10 \text{ sq. in.}}{9 \text{ in. wide}} = 1 \cdot 11$.

Thickness say . . . . . $1\frac{1}{8}$ inches.

Decimals are handier in calculation than fractions, but the final dimensions should be in feet, inches and fractions, to suit the workmen who make the girder.

SECTIONAL AREA OF TOP FLANGE.

Theoretically cast iron is six times stronger in compression than in tension, but if the areas of the flanges correspond to these proportions any defects in the compression flange, such as honeycombing, &c., would be six times as detrimental as in the tension flange, and hence the proportion usually adopted in practice is 4 to 1.

$\frac{10}{4}$ = ... 2·5 sq. inches.

SIZE OF TOP FLANGE.

The width of compression flange should not be less than $\frac{1}{60}$ clear span, or there will be risk of bending laterally; it should also not be less than $\frac{1}{5}$ width of bottom flange. The thickness may be $\frac{1}{3}$ to $\frac{1}{4}$ the width; if too thin there will be risk of failure by the outer edge buckling or crumpling.

$\frac{1}{60}$ of 120″ = 2″ minimum, $\frac{1}{5}$ of 9″ = 1·8″ minimum.

Width say . . . . . 3 inches.

$\frac{1}{3}$ of 3 = 1, $\frac{1}{4}$ of 3 = ·75,

but 2·5 sq. inches area ÷ 3 inches width = ·833.

Thickness say . . . . . $\frac{7}{8}$ inch.

In using a cast-iron girder for carrying a load on the top flange, care must be taken that the load rests evenly upon it. When used as a bressumer for carrying a brick wall, a course of 2-inch York stone templates is generally interposed between the top flange and the wall to make up the width, and in such a case there is much risk of unequal loading, so that it will be safer to add a trifle to the thickness of top flange and make it

say . . . . . 1 inch.

When the load is carried by the bottom flange on one side, there will be greater risk of failure by lateral bending, and then it may be desirable to increase the width of top flange instead of thickness.

WEB.

In a cast-iron girder the thickness of web is made proportional to the thickness of flanges, and not to the shearing stress, being always in excess of the latter requirement. The web should be $\frac{2}{3}$ to $\frac{3}{4}$ the thickness of bottom flange, and tapered throughout its depth towards the top.

$\frac{2}{3}$ of 1·125 = ·75, $\frac{3}{4}$ of 1·125 = ·844.

Thickness say . . . . . $\frac{3}{4}$ inch at top. $\frac{7}{8}$ inch at bottom.

Castings should, as far as possible, be designed to cool uniformly, and without any great variation in thickness of the different parts, to prevent initial strains being set up in

cooling, hence the foregoing limits. Theoretically, the limits might be greatly extended, but the various contingencies of practical work require the designer to keep well on the safe side.

SHEAR STRESS ON WEB.

With a uniformly distributed load the shear stress at each abutment is equal to half the total load, diminishing as ordinates to a triangle, and disappearing entirely at the centre of span. Cast iron may be subjected to 2·5 tons shear stress per sq. inch of section, but in this web we have say $11 \times \frac{13}{16} = 8{\cdot}94$ sq. inches area to resist a maximum stress of $\frac{10{\cdot}5}{2} = 5{\cdot}25$ tons, being only $\frac{5{\cdot}25}{8{\cdot}94} = {\cdot}587$ of a ton per sq. inch.

STIFFENERS OR FEATHERS.

These are to prevent buckling, and should generally be even in number to avoid a centre one, about equally divided over girder, 4 to 5 feet apart, same thickness as web, and carried out to extreme edge of each flange. The outline in cross section of girder may be straight, but is better when slightly hollowed. Sometimes small intermediate feathers 3 inches or 4 inches deep are put in. There will always be a stiffener closing up each end of a parallel girder. In this case say one at each end, and two others 2 feet from centre line on both sides of the girder.

ANGLE FILLETS.

Curved fillets, or "featherings," should be put in all angles, to a radius of $\frac{1}{2}$ to $\frac{2}{3}$ thickness of parts joined.
Say throughout this girder . . . $\frac{5}{8}$ inch radius.

The object of these fillets is to avoid an abrupt change of direction in the casting, where the arrangement of the crystals in cooling would give a line of weakness. Architects frequently design castings of all kinds with plain angle fillets instead of curved ones, but it is an objectionable practice. The weight of curved fillets is approximately,

Inches radius^2 = lbs. per foot run.

DEFLECTION.

By Molesworth (p. 169, 21st edition) the safe deflection of a cast-iron girder is $\frac{1}{40}$ inch per foot span under a test load of $\frac{1}{3}$ breaking weight, but this is considered to be rather a full allowance.

$\frac{10{\cdot}5 \times 1}{40} = {\cdot}2625$, say . . . . . $\frac{1}{4}$ inch.

A formula based upon experiment is as follows :—

W = gross distributed load in tons.

s = effective span in feet.

d = mean depth in inches.

$$\text{Deflection} = {\cdot}0005 \frac{W s^2}{d}$$

$$\doteq {\cdot}0005 \frac{11{\cdot}15 \times 10{\cdot}75^2}{12} = {\cdot}054, \text{ say} \quad . \quad . \quad \tfrac{1}{16} \text{ inch.}$$

CAMBER.

Camber is always given in excess of the calculated deflection, in order that the girder may not come down to a horizontal line, which invariably appears to sag or drop in the centre. Usual camber for a cast-iron girder, $\frac{1}{2}$ to $\frac{3}{4}$ inch per 10 feet clear span,

say . . . . . $\frac{1}{2}$ inch.

AREA OF STONE FOR BEDDING GIRDER.

$9'' \times 9'' = 81$ sq. inches, each bearing surface of girder

$\frac{81 \times 12 \text{ tons on stone}}{4 \text{ tons on brick}} = 243$ sq. inches area of each stone.

WIDTH OF STONE.

9 inches bearing of girder + say $4\frac{1}{2}$ inches margin beyond end of girder = ... $13\frac{1}{2}$ inches.

LENGTH OF STONE.

$\frac{243 \text{ sq. inches area}}{13\frac{1}{2} \text{ inches width}} =$ . . . . . 18 inches.

These are suitable dimensions, because they are multiples of a half brick, $4\frac{1}{2}$ inches.

CHAMFER AT EDGE OF BEARING.

In order to prevent the stone from spalling or chipping by the load coming too near the edge, it is well to chamfer the edge. With small loads the stone may project an amount equal to the chamfer, so that the effective span is not increased. For small spans the chamfer should be about $1\frac{1}{2}$ inches per 10 feet clear span.

Say . . . . . $1\frac{1}{2}$ inches.

THICKNESS OF STONE

generally varies from $\frac{1}{3}$ width of girder to $\frac{1}{2}$ length of stone.

$\frac{1}{2}$ of $9 = 4\frac{1}{2}$, $\frac{1}{2}$ of $18 = 9$,

say . . . . . 6 inches.

SIZE OF STONE.

Each stone will therefore be $13\frac{1}{2} + 1\frac{1}{2}$" chamfer = 15" wide × 18" long × 6" thick.

GIRDER PADS.

A pad of tarred roofing felt should be laid under each end of girder to counteract the roughness of the casting and secure an even bedding.

ESTIMATE OF WEIGHT.

Taken at $37\frac{1}{2}$ lbs. per foot super 1 inch thick.

Top flange	11′ 6″ × 3″ × 1″	=	109 lbs.
Bottom flange	11′ 6″ × 9″ × $1\frac{1}{8}$″		363 „
Web	11′ 6″ × 11″ × $\frac{13}{16}$″		322 „
Stiffeners	8 (11″ × $2\frac{1}{2}$″ × $\frac{13}{16}$″)		46 „
Fillets, say	4 × 11′ 6″ 12 × 11″ 12 × 1″ 12 × 4″ }	62 feet at $\frac{3}{8}$ lb.	23 „
			863 lbs.

$\frac{863}{2240} = \cdot 385$ of a ton, so that our approximate allowance of ·4 of a ton was quite sufficient, and the design may now be prepared, but had the estimate of weight from dimensions come out very different from the assumed weight, it would have been necessary to revise the calculations before proceeding with the design.

ESTIMATE OF COST, INCLUDING ORDINARY PROFIT.

	£	s.	d.
Iron and labour ·385 ton at £7 10s. ..	2	17	9
Hoisting and erection „ 15s. ..	0	5	9
Total cost, fixed..	£3	3	6

Exclusive of any brickwork or masonry, of cost of pattern, or of any charge for testing. The cost will, of course, vary with the market value of iron and labour, and with the number of girders to be supplied.

Approximate cost of cast-iron parallel girder in £ = ·035 W l = ·035 × 10 tons × 10 feet = . . . £3 10s.

For design see Plate 1, Figs. 2, 3 and 4.

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Fig

OF

CHAPTER II.

FLITCHED BEAM.

(See Plate 2.)

REQUIRED a flitched beam of Quebec oak, to carry a distributed dead load of half ton per foot run, over a 15-feet opening.

A flitched or sandwich beam generally consists of two pieces of timber bolted together with a wrought-iron plate between, as Fig. 5. When greater strength is required than a single plate will give, it may consist of one piece of timber with a wrought-iron plate on each side, as Fig. 6, or of three pieces of timber with two plates, as Fig. 7; or it may be of one piece with a wrought-iron plate on the bottom, as Fig. 8. The latter is the best in principle, owing to the iron being then under tensile stress only, but it requires firmly securing to the timber to prevent any sliding or yielding at the bottom, and hence should have angle irons riveted at each end to form joggles or cleats sufficient to resist the horizontal shear, as A B in Figs. 9 and 10, in addition to bolts or coach screws to hold it up. Rolled joists have been used instead of flitch-plates, as in Fig. 11, but if so much strength is requisite, and rolled joists can be obtained, timber should not be a component, unless for use in shop-fronts, where the wood is useful for fixing facias to, &c.

We will assume Fig. 5 to be the pattern required, provided that the calculations show it to be suitable.

CLEAR SPAN.

Say . . . . . 15 feet.

EXTERNAL LOAD.

½ ton per foot run over 15 feet span = ·5 × 15,
say . . . . . 7·5 tons.

APPROXIMATE WEIGHT OF BEAM.

$\frac{W l}{120} = \frac{7{\cdot}5 \times 15}{120} =$ . . . . . ·94 ton.

TOTAL LOAD ON BEAM.

From external load 7·5 tons, from weight of structure ·94 ton, 7·5 + ·94 = 8·44, say . . . 8·5 tons.

ORDINARY FORMULA FOR FLITCHED BEAM.

(See Hurst's Handbook, p. 26.)

b = total breadth inches, d = depth inches, t = thickness flitch inches, L = length of bearing [span] in feet, W = breaking weight in cwts. [in centre].

$$W = \frac{d^2}{L}(C\,b + 30\,t)$$

C = 4·0 Teak.
= 3·7 Oak, English or Baltic.
= 3·2 Oak, Canadian (Quebec).
= 3·0 Northern pine, Memel and Dantzic.
= 2·8 American pine and spruce.
= 2·2 Elm.

Teak is suitable for all cases, but very expensive; oak for damp positions; pine and spruce for dry; and elm for wet situations. Their practical difference is more a question of durability than strength.

FACTOR OF SAFETY.

For timber in permanent structures the safe load should not exceed $\frac{1}{10}$ of the breaking weight; for temporary structures it may be as much as $\frac{1}{5}$ for dead loads or $\frac{1}{4}$ for live loads.

APPROXIMATE DIMENSIONS OF FLITCHED BEAM.

As the dimensions must be assumed and then proved by calculation to be sufficient, we may take

$$\sqrt[3]{(c \times \text{gross load tons distributed} \times \text{span feet})}$$

as giving roughly the width of side for a square flitched beam.

$c = 15$ for oak, 18 for fir.

$\sqrt[3]{15 \times 8{\cdot}5 \times 15} = \sqrt[3]{1912{\cdot}5}$ = say . . $12\frac{1}{2}$ inches.

THICKNESS OF FLITCH.

Should be about $\frac{1}{12}$ that of the wood for fir and pine, and about $\frac{1}{10}$ for oak (*Hurst*).

It is frequently less than this, and never exceeds 1 inch thick.

$\frac{12{\cdot}5}{10} = 1{\cdot}25$, but try, say . . . . 1 inch.

STRENGTH OF ASSUMED BEAM.

$$W = \frac{d^2}{L}(C\,b + 30\,t) = \frac{12{\cdot}5^2}{15}(3{\cdot}2 \times 12{\cdot}5 + 30 \times 1)$$

$$= \frac{156 \times 70}{15} = 728 \text{ cwt. in centre to break beam.}$$

$$\frac{728 \times 2 \text{ for load distrib.}}{20 \text{ cwt. in 1 ton} \times 10 \text{ fact. safety}} = 7{\cdot}28 \text{ tons safe load dist.}$$

This is not enough, and is due to the flitch plate being thinner than $\frac{1}{10}$ of the timber. We require 8·5, or about $\frac{1}{6}$ more, and might obtain it most economically by deepening the beam, but the flitch plate already seems unwieldy.

To avoid extra charges wrought-iron plates should not exceed 5 cwt. in weight, 30 sq. feet in area, 15 feet in length, or 4 feet in width.

The flitch plate would be about $17' \times 12\frac{1}{2}'' \times 1''$ or say 17 feet super, $6\frac{1}{2}$ cwt., 17 feet long, exceeding two of the limits, and particularly the limit of weight.

We cannot reduce the length and must be content to pay the extra charge for it. From the Cleveland district larger and heavier plates can be obtained than from the Staffordshire district, viz. up to 10 cwt., 20 feet long, 4 feet 6 inches wide (see p. 59 of the author's 'Handbook for Mechanical Engineers,' Spon, 6*s*.), but usually the Staffordshire limits given above should not be exceeded.

Under these circumstances it seems probable that two flitch plates would be more suitable. Fig. 6 is a very undesirable method, on account of the distance from centre to centre of the bolts, in the length of the beam, allowing the plates to buckle on the compression edge, which they are

unable to do when bolted between the timbers. Another objection would be that the timber would be used solid, while the ordinary method of sawing and reversing enables the quality to be seen and facilitates the seasoning, and reversing the ends equalises the strength in the event of local defects.

If, however, we cut the timber into three, and put the two flitch plates between, as Fig. 7, we obtain all the advantages and none of the disadvantages.

MODIFIED DESIGN.

Let us now take the oak in three pieces, each 4 inches wide by 12½ inches deep, and two flitch plates, each ⅝-inch thick by 12 inches deep. We cannot have the plates 12½ inches deep without paying extra (see p. 40), and the ¼-inch clearance given top and bottom will prevent any possibility of the beam resting on the iron only and damaging the supporting surface.

STRENGTH OF BEAM.

$$W = \frac{d^2}{L}(C\,b + 30\,t) = \frac{12 \cdot 5^2}{15}(3 \cdot 2 \times 13 \cdot 75 + 30 \times \cdot 625 \times 2)$$

$$= 848 \cdot 96.$$

$$\frac{848 \cdot 96 \times 2}{20 \times 10} = \quad . \quad . \quad . \quad . \quad . \quad . \quad 8 \cdot 49 \text{ tons,}$$

which appears to be just right.

BEARING SURFACE.

The working pressure on timber across the grain must not under any circumstances exceed

250 lbs. per sq. inch for fir;
350 „ „ oak.

$\frac{8 \cdot 5 \times 2240}{2} = 9520$ lbs. pressure at each end, $\frac{9520}{350} = 27 \cdot 2$ sq. inches area. Width of oak 12 inches, $\frac{27 \cdot 2}{12} = 2 \cdot 27$ inches length of bearing necessary, but as a rule it should not be less than $\frac{2}{3}$ the width.

$12 \times \frac{2}{3} =$ . . . . . 8 inches

SUPPORTS FOR BEAM.

The ends of the flitched beam should rest on York stone templates $2\frac{1}{2}$ inches or 3 inches thick; they may be coped out of a piece of York paving. Load on each bearing surface $= \frac{8 \cdot 5}{2} = 4 \cdot 25$ tons, therefore the size of stone need only be just over 1 sq. foot area to limit the pressure upon the brickwork to 4 tons per sq. foot. The actual size of the stone will generally depend somewhat on its position. If the beam is placed over an opening in a wall, the wall should not be less than $1\frac{1}{2}$ bricks thick, and each template would then be $13\frac{1}{2}$ inches square. If the beam is placed between two walls and at right angles to them the templates might be, say $13\frac{1}{2}$ inches deep and 18 inches on face, supported by brick corbels as shown.

TOTAL LENGTH.

Span 15′ 0″ + 2 bearing surfaces, 1′ 4″ = 16 feet 4 inches.

BOLTS FOR BEAM.

There is no rule for the size or spacing of the bolts, generally two would be put in a vertical line at each end, and the remainder spaced zigzag throughout the beam about

3 inches from the edges and 2 feet apart, but avoiding, if possible, the placing of a bolt on the lower side of the centre of beam. The diameter should never be less than $\frac{5}{8}$-inch, nor under ordinary circumstances greater than 1 inch; $\frac{5}{8}$-inch and $\frac{3}{4}$-inch may be considered as the usual sizes.

As a rule the diameter of the bolts should not be less than $\frac{1}{8}$ inch more than thickness of flitch to allow of the holes being punched instead of drilled, to save expense.

Diameter say, $\frac{5}{8} + \frac{1}{8} =$ . . . . . $\frac{3}{4}$ inch.

ACTUAL WEIGHT OF BEAM.

	lbs.
Oak at 56 lbs. per cubic foot, say . .	960
Iron at 480 lbs. „ „ . .	820
Bolts, nuts and washers	108
	1888

$\frac{1888}{2240}$ = say . . . . . ·84 ton,

which is on the safe side of the assumed weight.

CHECK ON DIMENSIONS.

Before proceeding with the design it may be well to check the dimensions by a more precise method than the ordinary formula, viz. by the modulus of section, which presents no difficulty.

For explanation of modulus of section see p. 75 *et seq.*

Modulus of section of solid rectangle $= \frac{bd^2}{6} = Z.$

Modulus of rupture, wrought iron, 42,000 } = C.
„ „ American oak, 10,600 }

Coefficient of reaction (see p. 7) = k.

$$W = Z C \frac{k}{l}.$$

$$3\left(\frac{4 \times 12 \cdot 5^2}{6}\right) \times 10{,}600 \times \frac{8}{15 \cdot 67 \times 12} = \begin{cases} 140{,}947 \text{ lbs. for} \\ \text{the wood;} \end{cases}$$

$$2\left(\frac{\cdot 625 \times 12^2}{6}\right) \times 42{,}000 \times \frac{8}{15 \cdot 67 \times 12} = \begin{cases} 53{,}606 \text{ lbs. for} \\ \text{the iron;} \end{cases}$$

or together 140,947 + 53,606 = 194,553 lbs., which at 2240 lbs. = 1 ton, and factor of safety of $\frac{1}{10}$ =

$$\frac{194{,}553}{2240 \times 10} = \quad . \quad . \quad . \quad . \quad . \quad . \quad 8 \cdot 7 \text{ tons}$$

Showing that the beam has rather more than the required strength.

COMPLETE DESIGN.

The complete design may now be proceeded with, see Figs. 12 and 13, Plate 2.

PAINTING.

The separate pieces of wood and iron should be painted all over with thick red lead or oxide paint before being put together, and the bolts dipped while hot in boiled linseed oil. After fixing, the whole should be painted two coats best oil colour and finished approved tint. Should any moisture get to the unprotected iron in contact with the oak, black ink stains will be caused from the production of tannate of iron,

COST OF BEAM.

	£	s.	d.
Am. oak, 16′ 6″ × 12½″ × 12½″ = 17·8 cubic feet at 160*s.* per load of 50 feet .	2	16	6
16′ 6″ × 12½″ × 2″ = 34 feet super sawing, at 9*s.* per 100 feet super . . .	0	3	1
3 × 21 perforations for bolts to templates and fixing same, at 1*d.* . . .	0	5	3
Wrought iron, in flitch plates, 2 × 16′ 6″ × 12″ × ⅝″ = 33 feet super, at 25 lbs. per foot super = 825½ lbs. = 7 cwt. 1 qr. 13 lbs., at 7*s.* 6*d.* per cwt.	2	15	2
2 × 21 holes punched to template, at 1*d.*	0	3	6
Bolts, nuts and washers (21 × 1 1/12 foot × 1·5 lbs.) + (21 × 1 lb. for head nut and two washers) = 55·125 lbs., say ½ cwt., at 22*s.* 6*d.* per cwt.). . .	0	11	3
Painting—			
Priming coat on wood and iron = 23 yards super, at 4*d.* per yard . . .	0	7	8
Two coats oil, 10 yards super, at 6*d.* per yard	0	5	0
Hoisting and erection, say . . .	0	12	6
	7	19	11

Total cost, say £8.

MAINTENANCE OF BEAM.

The nuts should be tightened up at intervals, particularly after a dry summer, in order to compensate for the shrinkage of the wood and to maintain the friction between the wood and iron.

CHAPTER III.

RIVETED JOINTS.

(See Plate 3.)

RIVETS are made with a shank and one head, Fig. 14, and after being heated and put in place a second head is formed at the opposite end, Fig. 15, which may or may not be the same shape as the original head. In girder work the cup, snap or button head, Figs. 14 and 15, is usual for both ends, except on bearing surfaces where the second head is countersunk, Fig. 16.

In tank and boiler work the original heads, if not cup heads, are cheese or pan heads, Fig. 17, and the second heads are generally snap heads for either hand or hydraulic riveting, but may be point or cone heads, Fig. 18, for hand work. Good rivets, especially long ones, should have a small curve under the head as shown in Fig. 14, and the sharp edge taken off the hole to match, the heads are then less likely to fly off in cooling after being hammered up. Snap head rivets vary in shape; hemispherical are not used at all, as they appear very large and metal would be wasted in them. The largest snap heads are set off as in Fig. 19, the diameter of shank being used as radius. Fig. 20 shows a good proportion, being comparatively thin in the centre and thick at the edges; this is started by dividing the diameter into three equal parts. Fig. 21 is easily set off but is rather too thin at the edges.

In ship work, to avoid the thin edge of countersunk rivets

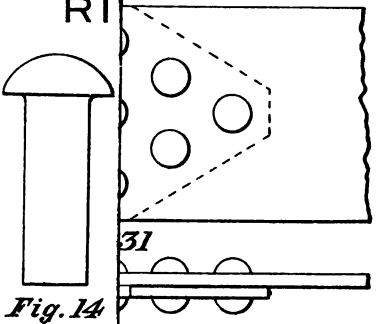


Fig. 14

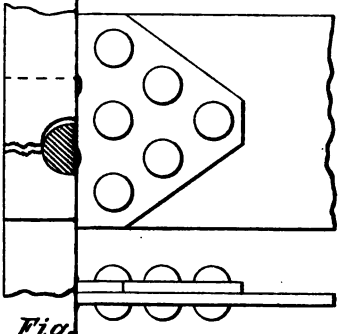
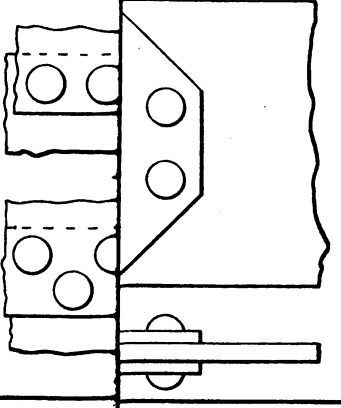


Fig.



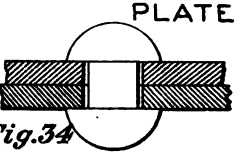


Fig.34

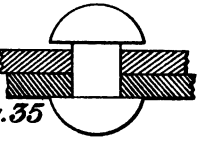


Fig.35

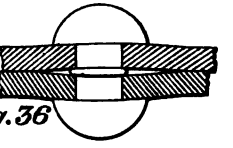


Fig.36

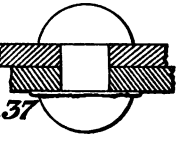


Fig.37

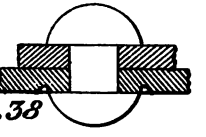


Fig.38

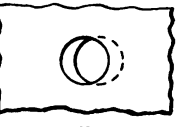


Fig.39

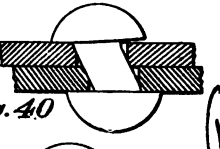


Fig.40

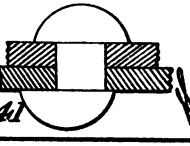


Fig.41

E. & F. N. Spon, London

PHOTO-LITHO SPRAGUE & Co LONDON.

as Fig. 16, the holes are sometimes countersunk with a shoulder as Fig. 22, and the rivets are also "left full on the countersink" as shown.

The diameter of head averages 1·7 times that of shank, with a thickness equal to $\frac{3}{4}$ of the shank.

MODE OF FAILURE.

A riveted joint may fail in various ways :—

1. By tearing through one of the plates in the line of rivet holes, as Fig. 23.

2. By crushing the rivets against the edge of plate in the rivet holes through deficiency of bearing surface, as Fig. 24.

3. By the wedge action of the rivets tearing the plate through insufficient lap, as Fig. 25.

4. By shearing the rivets through the joint between the plates, as Fig. 26.

A riveted joint, as Fig. 31, may fail (*a*) by tearing through the plate at the first rivet hole; (*b*) by tearing through the second line of holes and shearing the first rivet, or by tearing through any line of holes and shearing the intervening rivets ; (*c*) by shearing all the rivets or (*d*) by crushing the rivets against the edge of plate in the rivet holes. All these points require attention in a properly designed joint.

In a continuous joint, as Fig. 27, if the lap be insufficient failure may occur ; (*e*) by tearing through each rivet hole.

FORM OF RIVETED JOINTS.

When the object of riveting is to connect two or more plates to increase the area, as in tank or boiler work, they

may be single riveted, as Fig. 27, or double riveted, as Fig. 28. The increased pitch of the latter enables a greater sectional area of plate to remain for resisting the stress, and provides a greater number of rivets in a given length to be sheared through before failure can occur.

When two or more rows of rivets are required they may be chain riveted as Fig. 29, which is adopted chiefly in girder work; or they may be zigzag, reeled or staggered, as Fig. 30, which is generally used in boiler work.

When the ends of two bars or plates in tension are riveted together, the rivets may be chain or zigzag, but they should be so grouped as to reduce in number towards the outer ends of the plates, as in the lap joint, Fig. 31, the butt joint with single covers in Fig. 32, or the butt joint with double covers in Fig. 33. This arrangement of rivets gives maximum strength; for instance, if failure should take place through A A, Fig. 31, the full section of plate less one rivet hole would have to be torn asunder; through B B two rivet holes have to be deducted, but the rivet in line A A must also be sheared for failure to occur.

RIVET HOLES.

The nominal diameter of a rivet is the diameter before being heated, the hole is larger to allow of the heated and expanded rivet entering without difficulty. The hole, whether drilled or punched, averages $\frac{1}{20}$ inch larger in diameter (at the junction of the plates) than the rivet, and the finished rivet of course fills the hole.

"Usually for all rivets between $\frac{5}{8}$ inch and 1 inch diameter the holes are drilled $\frac{1}{16}$ inch diameter larger than the nominal size of rivet."—*Pendred.*

With some bridge builders, for $\frac{3}{4}$ inch riveting the holes would be punched $\frac{7}{8}$ inch diameter at small end and the rivets (nominally $\frac{3}{4}$-inch rivets) would be $\frac{11}{16}$ diameter, but the above is the usual rule.

LENGTH OF RIVET.

The total length of rivet shank to fill the hole and make the closing head is commonly made equal to the total thickness of plates passed through + ($\frac{1}{16}$ inch for each joint or $\frac{1}{32}$ inch for each plate) + $1\frac{1}{2}$ times its diameter. For instance, a $\frac{3}{4}$-inch rivet would have a snap head $1\frac{1}{4}$ inches diameter and $\frac{9}{16}$ inch thick, with a rivet hole $\frac{8}{10}$ inch diameter, and passing through three $\frac{1}{2}$-inch plates, would have a length from under side of head to point of $2\frac{3}{4}$ inches.

SELECTION OF DIAMETER.

For rivets passing through two thicknesses of plates

$$d = \tfrac{3}{4}t + \tfrac{5}{16} \quad \text{to} \quad \tfrac{7}{8}t + \tfrac{3}{8},$$

which for two $\frac{5}{8}$-inch plates would be

$$\tfrac{25}{32} = \tfrac{3}{4} + \tfrac{1}{32} \quad \text{to} \quad \tfrac{59}{64} = \tfrac{7}{8} + \tfrac{3}{64},$$

say nominally $\frac{3}{4}$ or $\frac{7}{8}$ rivets; and for several thicknesses $d = \frac{1}{8}\,T + \frac{5}{8}$, which for six $\frac{5}{8}$-inch plates would be $1\frac{3}{32}$, or say nominally, 1-inch rivets.

The usual diameter of rivets in hand riveting varies from $\frac{1}{2}$ inch to $\frac{7}{8}$ inch, in machine riveting they may be used up to $1\frac{1}{4}$ inch diameter.

Fairbairn's rule, which applied more particularly to boiler

and tank work, made the diameter of rivet equal twice thickness of plate for plates below ½ inch thick, and 1½ times thickness of plate for plates ½ inch thick or over.

TENSION IN RIVETS.

Professor Unwin states: "The tension in the rivet may be estimated at 21,000 lbs. per square inch of its section, and the friction due to this would be about 7000 lbs. per sq. inch of rivet section. Experiments show a still greater friction, but if the tension in the rivet exceeds the elastic limit, its performance cannot be relied upon. English engineers entirely neglect the friction in estimating the strength of the joint, the reasons assigned being that the amount of tension in the rivet is not ascertainable, and that vibrations and other causes, tending to slightly elongate the rivet, may, in course of time, destroy it altogether."

LAP AND PITCH.

The least lap and pitch, to prevent the possibility of the hole bursting through to edge of plate, or a crack developing between the holes, must allow a clear space of metal in each direction equal to the diameter of the rivet. Generally a joint with ¾-inch rivets will have a minimum of 1⅞ inch pitch and 2 inch lap in boiler work, and in girder work 3 inch pitch with a distance of 1⅛ inch from centre of rivet to edge of plate.

The proper pitch for girders will depend upon the horizontal shear stress in each case. This will be dealt with in the design for a wrought-iron girder.

ORIGINAL TENACITY OF MATERIAL.

Iron plates	. .	46,000 lbs.	per sq. inch.
Steel plates	. .	62,000	„ „
Rivet iron	. .	62,000	„ „
Rivet steel	. .	66,000	„ „

TENACITY OF PLATES AFTER DRILLING AND PUNCHING.

Drilled—increased by 10 or 12 per cent.

because, of the net area left, no selection of the weakest place is possible, and for other reasons.

Drilled holes should have the edges chamfered to remove the burr.

Punched (soft iron)		decreased by	5 to 10 per cent.
Do.	(hard iron)	„	15 to 30 „
Do.	(mild steel)	„	8 to 35 „

but the loss in punching is restored if the hole be rimered out to $\frac{1}{10}$ inch larger diameter.

In punched plates the small sides of the holes should come together, particularly for flush rivets, or rivets with small heads. The punching generally causes a slight bending of the plate, which is not objectionable unless the joint is so hollowed as to endanger the formation of a collar on the rivet shank.

Drilled holes cost £1 per ton more than punched holes, and the quality of the iron is not so readily shown as it is by the punchings.

AVERAGE TENACITY OF JOINTS.

in lbs. per square inch on net section.

	Iron Plates.	Steel Plates.
Single riveted, drilled .	40,500	62,000
„ „ punched .	35,400	55,800
Double riveted, drilled .	43,700	65,700
„ „ punched .	39,000	62,000

RESISTANCE TO SHEARING.

The shearing resistance of iron and steel bars averages $\frac{4}{5}$ of their tensile strength, but in the case of rivets the resistance is from various causes apparently less. The average resistance to single shear per square inch of section is

Iron rivets, punched holes . .	46,000 lbs.
„ „ drilled „ . .	43,000 „
Steel „ punched „ . .	53,000 „
„ „ drilled „ . .	49,000 „

so that a $\frac{3}{4}$ rivet in a punched hole in single shear would have an actual diameter of ·8 inch, an area of ·5 sq. inch, and a resistance of 23,000 lbs., which, at a factor of safety of $\frac{1}{4}$, would give a safe load of 5750 lbs., or 2·567 tons.

A common rule for safe shearing stress on bolts or rivets is

$$4\,d^2 = \text{load in tons single shear,}$$

whence the number of rivets required in a given case would be $= \frac{W}{4\,d^2}$, or the diameter for a given load and number $= \sqrt{\frac{W}{4\,n}}$.

In American practice a safe load for shear stress on rivets is considered to be 7500 lbs. per sq. inch, or say $\frac{1}{6}$ of the ultimate strength.

Rivets in double shear = $1\frac{1}{2}$ times strength in single shear.

CRUSHING ACTION IN HOLE.

Professor Unwin states with regard to this: "The value of the crushing stress, which produces injury to the tenacity or shearing resistance of the joint, is very uncertain. In the case of steel joints there is no indication of injury with crushing pressures of 50 tons per sq. inch. With the ordinary proportion of rivets, the crushing action will in no case need to be considered in single shear joints, and only in double shear joints when the plates are less than $\frac{7}{8}$ inch thick." On the whole Professor Unwin is "inclined to believe that the importance of crushing action has been exaggerated."

BEARING AREA.

The pressure upon bearing area of rivets, bolts and pins, measured by diameter of rivet × thickness of plate, should not exceed $7\frac{1}{2}$ tons per sq. inch, but unless there is any special reason for permitting a maximum, the pressure should be limited to 5 tons per sq. inch.

In American practice the bearing pressure is limited to 15,000 lbs. per sq. inch = 6·7 tons, which for a $\frac{3}{4}$-inch rivet in a $\frac{5}{8}$-inch plate would be 4218·75 lbs., or say 1·88 tons instead of 2·567 tons as found by shearing resistance only.

SPECIFICATION FOR GOOD RIVET IRON.

Best Lowmoor, Farnley or other Yorkshire iron, with a tensile strength of 25 tons per sq. inch, an elongation of 10 per cent in a length of 8 inches under a stress of 20 tons per sq. inch, and a contraction of 30 per cent. in area at point of fracture.

PRACTICAL RIVET TESTS.

Rivet shank to be doubled close when cold, and also when hot, without showing any sign of fracture. To bear a hole punched through it when hot with a round taper punch, and enlarged to diameter of shank without showing signs of cracking or splitting. The head when hot to bear hammering down to $2\frac{1}{2}$ times diameter of shank or to $\frac{1}{8}$-inch thick without fraying or cracking at the edges.

DEFECTIVE RIVETING.

If the points are not cooled before driving, there is risk of the head being formed before the rivet fills the hole, leading to permanent set and other difficulties when the load is applied, Fig. 34.

If the riveting is commenced before the plates are properly closed, the adjoining rivets may pull the plates together, and leave the first rivets loose, Fig. 35.

If the plates are not drawn close before commencing, there is risk of a collar being formed between them, on the shank of the rivet, Fig. 36.

The rivets should be of the proper length to suit the plates passed through, or there will either be too much material, leaving a frill as Fig. 37, or too little, leaving an imperfect head, or permitting the snap to cut into the plate as Fig. 38.

If the holes are not carefully placed to template, some will be partly blind, as Fig. 39, necessitating drifting and damaging the holes, and leading to the probability of the rivets not properly filling them, Fig. 40.

If the holder-up fails to drive the rivet home and keep it there while the riveters strike it, the head may be left projecting as Fig. 35.

If the riveters strike more on one side than the other, or in machine riveting, if the cups be not square with the work, or in line with the axis of rivet, the head may be formed on one side out of line, as Fig. 41.

All riveting should commence at the outer ends to drive the butt joints close and maintain the stiffness of the work.

Loose rivets, called "clinkers," are discovered by sounding with the hammer, or in old structures by the oozing of oil or moisture, and should be cut out and removed.

Cutting off rivet heads and punching out the shanks always strains adjoining rivets, and must therefore be done judiciously. In boiler work the defective rivets must be drilled out.

COST OF RIVETING.

A gang of riveters consists of three men and two boys. Two of the men are riveters and one a holder-up, one of the boys is a forge boy and the other the carrier, the latter is the younger. The riveters are paid 5*s.* to 6*s.* per day, according to locality, the holder-up 4*s.* to 4*s.* 6*d.*, boys 1*s.* to 2*s.* The

average wages of the gang may be taken as £1 per day. The average cost per rivet for piece work is as follows: ½-inch, 1*d.*, ⅝-inch, 1½*d.*; ¾-inch, 1½*d.*; London prices all through will be somewhat higher. An efficient gang can put in 40 ¾-inch rivets per hour if the platers have the work ready prepared for them.

FIELD RIVETING.

In outdoor work the rivets are heated by putting them through holes in a plate placed over the hollow of a fire in a small rivet forge, so that the shanks may be made red hot while the heads are kept cooler. Care has to be taken that the rivets are not over-heated or burnt; the forge boy is responsible for this. The platers connect by plater's bolts the parts which are to be riveted together, drawing them as close as possible. In the case of an open joint the riveters flog the plates, that is hammer them close to prevent a collar being formed on the rivet. The forge boy picks the rivets one at the time out from the perforated plate, as the riveters are ready, throws it on the ground to knock the clinker off, and the carrier then runs with it to the holder-up and dips the point in water before throwing it down to him. The holder-up then picks up the rivet, pushes it through the hole, and holds the dolly against it, the riveters then strike on the point, and the first effect is to swell out the middle of the rivet, where it is hottest, to fill the hole; by that time the heat has crept up to the point, and the force of the hammer knocks it down into a roughly rounded shape. One riveter then holds the snap or cup tool, on the roughly formed head, while the other strikes with the hammer to give it a perfect shape.

SHOP RIVETING.

Shop or yard riveting is now mostly machine work, either steam or hydraulic, and only the unavoidable junctions of parts separated for facility of carriage are made in the field or on the site. There is no difference in the general appearance between machine and hand made rivets except that in machine rivets there is more often a frill left round the new head. On cutting open and examining the joint, a machine rivet shows the hole to be much better filled, and testing the two modes of riveting, it is found that machine riveting is stronger because of the greater sectional area of the rivet, and the better grip upon the plates. The only disadvantage in machine riveting is a greater tendency to burst the holes from the fluid pressure generated at the centre of the rivet, the remedy for this being not to make the rivets quite so hot, and it is essential to keep the cup tool of the machine upon the rivet for a few seconds after the rivet is made, in order that a tight joint may be made.

CHAPTER IV.

WROUGHT-IRON GIRDER.

(See Plate 4.)

Detailed calculations and notes with design for :—

A wrought-iron (*a*) plate girder (*b*), supported at the ends (*c*), with parallel flanges (*d*) of uniform strength (*e*), to carry a uniformly distributed (*f*) dead load (*g*) of 1¼ tons per foot run (*h*) over a clear span (*i*) of 20 feet.

The girder to rest upon "girder beds" or templates of hard sandstone (*j*) supported by brickwork in mortar.

(*a*) See (*a*) p. 1.

(*b*) A plate girder, as generally understood, consists of top and bottom flanges, each formed of one or more plates, connected by angle-irons to a solid web, or web formed of plates. Other wrought-iron girders are known as rolled joists, lattice girders and box girders.

(*c*) See (*b*) p. 1. Wrought-iron girders are frequently used as continuous girders, and owing to the tensile and compressive strengths being more nearly equal in wrought iron than cast, there is less difficulty in proportioning the material to the stress at different points.

(*d*) See (*c*) p. 2.

(*e*) When a girder is made of equal and parallel plates throughout, it is said to be of uniform section; in girders of uniform strength the section varies proportionally with the stress.

PLATE 4

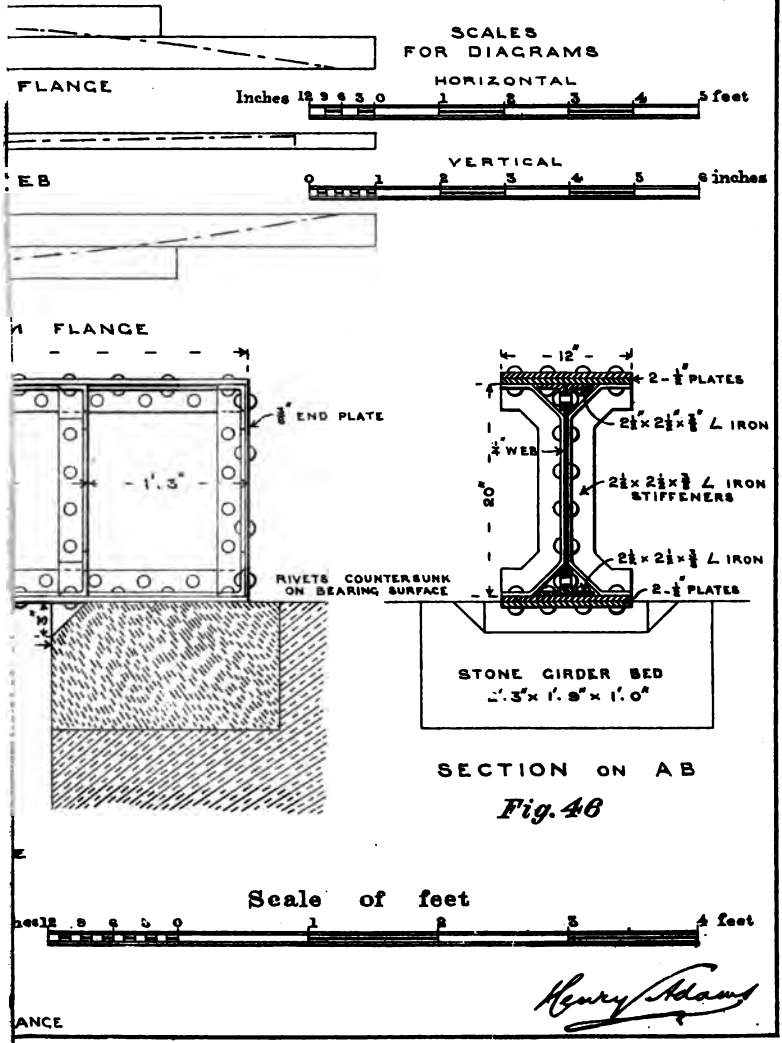

SCALES FOR DIAGRAMS
HORIZONTAL
Inches 12 9 6 3 0 1 2 3 4 5 feet
VERTICAL
0 1 2 3 4 5 6 inches
FLANGE
EB
FLANGE
- 1'. 3" -
⅜" END PLATE
RIVETS COUNTERSUNK ON BEARING SURFACE
- 12" -
2-½" PLATES
2½" × 2½" × ⅜" ∠ IRON
½" WEB
20"
2½" × 2½" × ⅜" ∠ IRON STIFFENERS
2½" × 2½" × ⅜" ∠ IRON
2-½" PLATES
STONE GIRDER BED
2'. 3" × 1'. 9" × 1'. 0"
SECTION ON AB
Scale of feet
12 9 6 3 0 1 2 3 4 feet
ANCE

Fig. 46

Henry Adams

PHOTO LITHO SPRAGUE & Co LONDON

(*f*) See (*f*) p. 2.

(*g*) A live load is considered to distress the material more than an equal dead load, on account of the vibration set up, but in ordinary cases no difference is made between the two classes of loads in calculating a wrought-iron girder. For railway girders the total load may generally be taken as $\frac{3}{4}$ rolling and $\frac{1}{4}$ dead load.

(*h*) A distributed load should always be given per foot run, so that allowance may be made for the portion carried by the girder immediately over the bearing surfaces.

(*i*) See (*g*) p. 3. The description and limiting dimensions here following apply equally well to larger girders, say up to 50 or 60 feet span, beyond which plate girders are not found economical.

(*j*) The stone beds at each end are usually chamfered, and may project an amount equal to the chamfer or may be flush with the brickwork; the latter is preferable in heavy girders, so that the load is thrown well over the body of the brickwork.

CALCULATIONS.

CLEAR SPAN.

The nominal clear span of a bridge girder is the width between the vertical faces of the abutments, the actual clear span will be from edge to edge of bearing surfaces, and the effective span will be the distance from centre to centre of the bearing surfaces.

Clear span . . . . . 20 feet.

DEPTH OF GIRDER.

This is measured in three ways; the depth of the web plate is the *net* depth of the girder, the distance between the centres of gravity of the flanges is the *mean* depth, and the distance from outside to outside of the flanges is the *gross* depth. One-twelfth of the effective span is the most economical mean depth. The depth of the web is usually made from $\frac{1}{8}$ to $\frac{1}{15}$ of the clear span, commonly about $\frac{1}{12}$, when the span in feet equals the depth in inches; but in small girders it should be made an even multiple of 2 inches, in order that market widths of plates may be used, to avoid the extra cost of shearing. Market widths run every 2 inches up to $\frac{7}{16}$ inch thick, above that every 4 inches.

Clear span, 20 feet ∴ depth of web . . . 20 inches.

In small girders, when the angle iron is considered as forming part of the flanges, one wing of each is included in the area and the depth of web taken as representing the mean depth.

WIDTH OF FLANGES.

The flanges, sometimes called booms or tables, should be from $\frac{1}{20}$ to $\frac{1}{40}$ of the span in width, or from $\frac{1}{2}$ to $\frac{1}{3}$ the depth of web; but room must be allowed for attaching the ends of stiffeners when they are bent round to the flanges. In this case 12 inches will probably do, but it is subject to modification when the sectional area is computed. In fact, all the calculations must be looked upon as tentative until their agreement with the subsequent calculations is proved. In large girders the figures are frequently revised before being finally settled upon.

Width say . . . . . 12 inches.

NET LOAD, OR LOAD OVER CLEAR SPAN.

1·25 tons × 20 feet . . . . . 25 tons.

WEIGHT OF GIRDER.

Upon one of the largest English railways the weight of a wrought-iron plate girder under distributed load is found as follows:—

W = total load in tons over clear span.

w = assumed weight of girder in tons (say $\frac{1}{4}$ W).

l = clear span in feet.

d = depth of girder web in feet.

s = stress in either flange in tons.

x = working stress in tons per sq. inch.

a = sectional area of one flange in sq. inches.

c = constant = $3\frac{1}{2}$ or 4 according to proportion of depth to span. It is the ratio of the total sectional area of girder (including equivalent of stiffeners) to the sectional area of one flange, and increases as the ratio of depth to length increases, because with greater depth the flange area will be reduced and the web area increased.

Then

$$\frac{(W + w)\,l}{8\,d} = s, \qquad \frac{s}{x} = a,$$

and

$$c\,a \times 12\,l = \text{cubic inches in girder,}$$

$$\frac{\text{cubic inches in girder}}{8000\ (= \text{cubic inches in 1 ton})} = \text{weight of girder in tons.}$$

This is equivalent to $\frac{W\,l\,R\,(44 - R)}{153{,}600}$, R being ratio of

span to depth, but the assumption of $\frac{1}{4}$ W for weight of girder is too much for spans below 100 feet, and too little for those above.

A rough approximation to weight of girder is obtained by taking

$$w = \frac{W\,l}{400},$$

and a better result, good enough for all practical purposes, is given by

$$w = \frac{W\,l}{375} \times \sqrt{\frac{l}{d}},$$

l being span in feet and d depth in inches.

$$\frac{25 \times 20}{375} \times \sqrt{\frac{20}{20}} = \quad . \quad . \quad . \quad . \quad . \quad 1{\cdot}3 \text{ tons.}$$

CHAMFER.

On edge of bearing surface to prevent chipping, say for York stone $1\frac{1}{2}$ inches per 10 feet clear span.

$$\frac{20 \times 1\frac{1}{2}}{10} = \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 3 \text{ inches.}$$

This may be reduced with large spans or light loads, and a better formula would perhaps be

$$\frac{\text{distributed load in tons} + \sqrt{\text{clear span feet}}}{10}.$$

$$\frac{25 + \sqrt{20}}{10} = 2{\cdot}95, \text{ say} \quad . \quad . \quad . \quad . \quad 3 \text{ inches.}$$

Granite may have half the amount of chamfer given to York stone.

LOAD TO EDGE OF BEARING SURFACES

= load per foot run × (clear span + chamfers when stone is flush with brickwork) added to weight of girder.

$$1{\cdot}25\ (20 + {\cdot}25 + {\cdot}25) + 1{\cdot}3 = 26{\cdot}925$$

say = ... 27 tons.

SAFE LOAD ON BEARING SURFACE.

The safe load on materials supporting ends of girders may be taken as follows:—

	Tons per foot super.		
Granite	15	=	$\frac{1}{100}$ ultimate.
Sandstone and best Portland ..	12	=	$\frac{1}{80}$ „
Limestone (ordinary)	9	=	$\frac{1}{40}$ „
And with stone template interposed:—			
Blue brick in cement	9	=	$\frac{1}{20}$ „
Stock „ „	6	=	$\frac{1}{20}$ „
„ „ lias mortar	5	=	$\frac{1}{10}$ „
„ „ grey lime mortar ..	4	=	$\frac{1}{4}$ „
And below the brickwork:—			
Cement concrete	4		
Lime concrete	2		
Natural compact earth	2		
Made earth rammed in layers ..	1		

In architectural work, with dead loads only, these figures might perhaps be somewhat increased, say up to 50 per cent. as a maximum, but for railway work, with engines weighing from 40 to 80 tons and running at 60 miles per hour, the factor of safety is not too great, considering the difficulty of

bedding truly and the irregularity of the mortar joints. Every care should be taken to avoid working the stone hollow or bedding it hollow, and the brickwork should be well grouted as it is carried up. In testing stone the blocks are dressed perfectly true and smooth, and placed between pieces of pine $\frac{3}{8}$-inch thick, so that the stone has every facility allowed for showing a high initial strength, which cannot be obtained in practical work.

When the height of a brick pier exceeds six times its least thickness, the above loads must be reduced to two-thirds.

LENGTH OF BEARING SURFACE.

l = total length of bearing surface in feet, half at each end.

b = breadth of bearing surface in feet, or width of bottom flange.

W = load to edge of bearing surface in tons.

w = load per foot run in tons.

s = safe load in tons per sq. foot on support.

Then by the method explained at p. 5.

$$l = \frac{W}{s\,b - w} = \frac{27}{(12 \times 1) - 1{\cdot}25} = 2{\cdot}5 \text{ feet},$$

half this being on each abutment, beyond the chamfers, say each end

$$\frac{2{\cdot}5}{2} = \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 1{\cdot}25 \text{ feet.}$$

By the common method of calculation no allowance is made for the load per foot run over the bearing surface itself,

and, as the load continues as far as the girder does, it is desirable to make provision for it as shown above. The difference in this case by the omission would be 1½ inches less on each bearing.

Girders over 50 feet span require cast-iron resting blocks at the ends over the stone, and over 100 feet span require roller bearings to allow for expansion and contraction due to changes of temperature.

EFFECTIVE SPAN

equals the distance from centre to centre of the bearing surfaces. Clear span, 20 feet + two chamfers ·5 feet, + one bearing surface 1·25 feet = effective span, say . 21·75 feet.

EFFECTIVE LOAD.

Effective span × load per foot run, added to weight of girder. (21·75 × 1·25) + 1·3 = say . . 28·5 tons.

STRESS IN FLANGES AT CENTRE.

Depth of web 20 inches + 1 inch assumed thickness of either flange = 1·75 feet mean depth of girder.

$$\frac{W l}{8 d} = \frac{28{\cdot}5 \times 21{\cdot}75}{8 \times 1{\cdot}75} = 44{\cdot}28$$

= say . . . . . 45 tons.

SHEARING STRESS.

With distributed load = nil at centre, increasing to half gross load at edge of bearing surface. Maximum stress

$\therefore = \frac{27}{2} =$ . . . . . 13·5 tons.

WEB.

In small girders the web is generally calculated for a direct shearing stress of 3 tons per sq. inch, or 2 tons per sq. inch when the depth exceeds 2 feet. It is not safe to allow more owing to the liability of web to crumple or buckle, and in large or important girders a more precise calculation must be made, taking a strip of the web as a column.

Maximum shear stress 13·5 tons, $\frac{13\cdot5}{3} = 4\cdot5$ sq. inches required in vertical section. Depth of web 20 inches, $\frac{4\cdot5}{20}$ = ·225 inch. maximum thickness, but no plate of a girder may be less than ¼ inch thick, therefore, say uniform thickness throughout = ... ·25 inch.

When depth exceeds 3 feet some engineers make ⅜ inch the minimum thickness of web, but this need not be if stiffeners are close enough and periodical inspection takes place to prevent corrosion.

When required to be safe against a possibly long-continued neglect, and particularly in positions exposed to moisture, no part of a wrought-iron girder should be less than ⅜ inch thick.

The horizontal sectional area of web should at least equal the gross sectional area of the two flanges at centre, but in small girders it will always exceed this, as the web plates exceed the theoretical thickness.

DIAMETER OF RIVETS.

All rivets in girders work usually ¾ inch diameter, up to maximum of 4 thicknesses of ½-inch plates, or 3 thicknesses

of $\frac{5}{8}$-inch plates. If the plates are $\frac{3}{4}$ inch thick the rivets should be $\frac{13}{16}$ or $\frac{7}{8}$-inch diameter. Rivets should never be less in diameter than the thickest plate passed through.

In this girder diameter say . . . . $\frac{3}{4}$ inch.

$\frac{3}{4}$-inch rivets require not less than $2\frac{1}{2}$-inch angle iron or 4-inch tee-iron.

PITCH OF RIVETS.

In small girders, or girders with thin plates, say up to $\frac{1}{2}$ inch thick, the pitch in compression flange is usually 3 inches. In girders with thicker plates 4 inches pitch may be used without fear of buckling. The pitch in tension flange is often 4 to $4\frac{1}{2}$ inches, but is sometimes made 6 inches as a maximum. Cover plates are generally riveted close pitch in order to save unnecessary length. Rivets on bearing surfaces are countersunk. Where uniform pitch throughout both flanges is desired, 4 inches is usual, except where cross girders are placed the multiple of a brick apart (9 inches), when $4\frac{1}{2}$ inches pitch is used.

The pitch at the ends of girder should be close enough in the tension flange to provide sufficient shearing area in the rivets connecting the web with flange angle irons to resist the horizontal shearing stress, which may be taken as equal to the vertical shearing stress at the same point. Thus shearing stress at abutment = 13·5 tons $\div \frac{20}{12}$ feet deep of web = 8·1 tons horizontal shear stress per foot run. $\frac{3}{4}$ rivets in single shear 2·5 tons, double shear $2{\cdot}5 \times 1\frac{1}{2} = 3{\cdot}75$. Then $\frac{8{\cdot}1}{3{\cdot}75}$ $= 2{\cdot}16$ rivets per foot run, and $\frac{12}{2{\cdot}16} = 5{\cdot}5$ inches maximum pitch. But $4\frac{1}{2}$-inch pitch will be safer and will work in better with a 3-inch pitch for the compression flange. The pitch is

sometimes altered at the ends of a girder to suit the length of bearing surface required, so as to finish evenly ; at other times the rivets are close pitched at the ends to meet the horizontal shear stress, and then wider pitched over the clear span.

If the rivets connecting web and angle irons are calculated for horizontal shear, there will always be sufficient area in rivets connecting angle irons and flange plates; but some engineers decide the pitch of the latter from the sectional area of the inner flange plate, providing at least an equal sectional area in the rivets between the termination of second flange plate and end of girder.

Up to 12 inches width of flange two rows of rivets are usually considered sufficient, extra rivets being put at covers and stiffeners. When exposed to the weather additional rows of rivets are desirable to close the plates and keep out moisture.

Top flange, compression, pitch	=	3 inches.
Bottom „ tension, „	=	4½ „

TOP FLANGE.

By Board of Trade regulations wrought-iron plate girders may be strained to 5 tons per sq. inch both in tension and compression, but wrought-iron is not so strong in compression as in tension, and 4 tons per sq. inch is therefore the usual limit of working stress in compression, no deduction being made for rivet holes, as the rivets are assumed to fill them.

The rivets should perfectly fill the holes in the finished work, or there will be a permanent set caused when the load comes on the girder, and an undue stress may be thrown on certain parts.

$\frac{45 \text{ tons stress in centre}}{4 \text{ tons per sq. inch}} = 11{\cdot}25$ sq. inches sectional area;

assumed width of flange 12 inches, $\frac{11{\cdot}25}{12} = {\cdot}9375$ inches thick

∴ say . . . 2 plates, each 12 inches × ½ inch.

The gross area of one side of each angle iron is frequently taken into account as forming part of the flange, in the present example the omission is made as being on the safe side and somewhat simplifying the work.

According to 'Notes in Building Construction' (Longmans) vol. iv. pp. 163–4, Prof. Unwin says the thickness of girder flange plates should not be less than $\frac{1}{700}$ of the span. This rule is however of very limited use, as it could only be applied to girders between 20 and 50 feet span.

Engineers generally prefer ½-inch and ⅝-inch plates for all cases, but in heavy work ¾-inch is not objected to. Some use, and others object to, plates having an odd sixteenth of an inch in the thickness.

Sometimes the bottom flange only is calculated and the top flange made equal to it, but as the circumstances are different it is safer to calculate both.

BOTTOM FLANGE.

Tension at 5 tons per sq. inch, deducting rivet holes, $\frac{45}{5}$ = 9 sq. inches net sectional area. Flange 12 inches wide, deduct two ¾ rivets through angle irons, and two ¾ rivets through edge of plates, 4 × ¾ = 3 inches, 12 − 3 = 9 inches net width, 9 sq. inches area ÷ 9 inches wide = 1 inch thick,

say . . . . 2 plates, each 12 inches × ½ inch.

Some engineers deduct 1 inch for every $\frac{3}{4}$ rivet if holes are punched, but this is not necessary when a fair margin is allowed throughout the other calculations. The hole for a $\frac{3}{4}$ rivet would generally be $1\frac{3}{16}$ inch bare.

Approximate area of flanges = $\frac{3}{8}$ W (W being net load in tons) which would give in this case 9·375 sq. inches, or rather more than the actual requirement.

TOTAL LENGTH OF GIRDER.

Should be a multiple of the pitch of the rivets, if the pitch is to be kept uniform throughout.

Clear span 20′ 0″ + chamfers 6″ + bearing surfaces 2′ 6″ together = ... 23′ 0″

In this case the $4\frac{1}{2}$-inch pitch for tension flange must be modified at the ends, as 23 feet is not a multiple of $4\frac{1}{2}$ inches, although it is of 3 inches.

The ends will be closed by $\frac{3}{8}$-inch plates, the same depth as the web, and covered top and bottom by the inner flange plates.

LENGTH OF OUTER PLATES OF FLANGES.

When there are two plates the outer one is about $\frac{2}{3}$ extreme length of girder; the exact length is found by the parabola in designing. In this girder each flange has two plates.

$\therefore 23 \times \frac{2}{3} = 15\cdot33$ length of outer plates,

say = ... 15 feet 4 inches.

DEFLECTION IN CENTRE UNDER FULL LOAD.

s = effective span in feet.

d = mean depth in inches (say web + top flange).

$$\frac{\cdot 015 s^2}{d} = \frac{\cdot 015 \times 21 \cdot 75^2}{21} = \cdot 338 \text{ inch,}$$

say . . . . . $\frac{3}{8}$ inch.

For railway girders the Board of Trade allows $\frac{3}{4}$ inch per 100 feet span ($= \frac{1}{1600}$) for deflection caused by maximum rolling load beyond the deflection due to maximum dead load.

CAMBER.

The usual camber is $\frac{1}{4}$ inch to $\frac{1}{2}$ inch per 10 feet clear span, but is sometimes made slightly in excess of the aggregate thickness of *short* plates and covers in bottom flange,

$$\text{say } \frac{20 \times \cdot 5}{10} = \text{ . . . . . } 1 \text{ inch.}$$

EXPANSION UNDER EXTREME TEMPERATURE.

Say $\frac{7}{16}$ inch per 100 feet.

$\frac{23 \times \frac{7}{16}}{100}$ = say $\frac{1}{10}$ inch, which is not worth considering, as ends of girders should always have $\frac{1}{4}$ inch clearance. Girders over 50 feet long should have a cast-iron slipper, very large ones have roller bearings. If ends are rigidly fixed, a difference of temperature of 75° F. will cause a stress of 5 tons per sq. inch, or 1 ton for every 15° F.

COVER PLATES TO JOINTS.

When the plates or angle irons run over the ordinary market dimensions,* butt joints must be made, covered by iron of same sectional area, and held by a sufficient number of rivets to give an equivalent sectional area on each side of joint. Staffordshire plates should not exceed 5 cwt., 30 sq. feet, 15 feet long, or 4 feet wide. Cleveland plates should not exceed 10 cwt., 60 sq. feet, 20 feet long, 4 feet 6 inches wide. Angle irons up to $4 \times 4 \times \frac{3}{4}$ may be had 30 feet long, and should not exceed 35 feet long. In this girder allow for joint in middle of web, and one in centre of first plate in each flange.

COVER PLATES TO WEB.

In a thin web these bear no proportion to the strain. Depth, say $20'' - 5''$ for angle irons = 15 inches. Width, say two $\frac{3}{4}$ rivets + four $\frac{3}{4}$ margins = $4\frac{1}{2}$ inches. Thickness, say $\frac{1}{4}$ inch.

$\therefore$ $=$ . . 2 plates, each $15'' \times 4\frac{1}{2}'' \times \frac{1}{4}''$.

Where the web plates vary in thickness the covers are made equal in thickness to the thicker of the two plates joined.

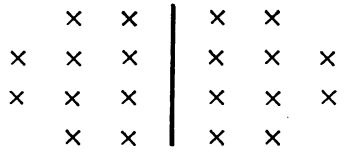
PITCH OF RIVETS IN WEB COVERS.

Rivets in each side of joint must be proportioned to the vertical shear stress. With a uniformly distributed load there is no shear stress in centre, therefore the pitch of the rivets in these covers may be made at pleasure, but usually made, say . . . . . 3 inches,

* See the author's 'Handbook for Mechanical Engineers' (Spon, 6*s.*), 3rd edition, p. 59.

COVER PLATE TO JOINT IN FIRST FLANGE PLATE.

Sectional area of part cut through = 12″ × ½″. Area of one ¾″ rivet = ·44 sq. inches; but worth say ·5 sq. inches, on account of the hole being punched slightly larger than the rivet, which the rivet afterwards fills. The friction between the plates, owing to the contraction of the rivets in cooling, is also worth something, but does not enter into the calculation. Net area of part cut through (12 inches – four ¾″ rivets) × ½″ = 9 × ½ = 4·5, and $\frac{4 \cdot 5}{\cdot 5}$ = nine rivets, as the minimum number in cover plate on each side of joint. To allow sufficient bearing area against the rivets in thin plates, the number of rivets on each side must always be greater than ⅚ of the net width of flange in inches. With a central web, the number must also be even. Pitch = 3 inches, say arrangement thus :—



making ten rivets on each side, and a total length of cover plate of 18 inches for top flange, and 25 inches for bottom flange owing to greater pitch; although when the plates in compression butt against each other and are planed at ends, there is theoretically no cover necessary on the compression flange. The outer plate of each flange must have the ends carried beyond the parabolic curve of stress (see Figs. 50 and 51) by an amount equal to half length of cover plate. The dotted lines C D on Figs. 50 and 51 show the theoretical termination of the plates, or where the parabola ends.

COMMON PROPORTIONS OF ANGLE IRONS.

$2 \times 2 \times \frac{1}{4}$	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	$4 \times 4 \times \frac{9}{16}$
$3 \times 3 \times \frac{3}{8}$	$4\frac{1}{2} \times 4\frac{1}{2} \times \frac{5}{8}$

also

$4 \times 3 \times \frac{1}{2}$	$6 \times 4 \times \frac{5}{8}$

but either size may vary in thickness from the preceding to the succeeding one.

SIZE OF ANGLE IRONS.

Say $\frac{1}{4}$ to $\frac{1}{5}$ width of flange, and thickness from $\frac{1}{6}$ to $\frac{1}{8}$ the width of sides, but width $-$ 3 times thickness must not be less than $1\frac{3}{4}$ times diameter of rivets.

$$\frac{12}{4} = 3, \qquad \frac{12}{5} = 2{\cdot}4, \qquad \text{say } 2{\cdot}5 \text{ width.}$$

And, to avoid crumpling of the compression flange, the plates should not project more than 8 times their least total thickness beyond the angle iron on each side.

$$\frac{12 - (2\frac{1}{2} + 2\frac{1}{2} + \frac{1}{4})}{2} = 3\frac{3}{8} \text{ actual projection.}$$

$\frac{1}{2}''$ plate $\times$ 8 $= 4''$ allowable projection, so that $2\frac{1}{2}$ will do for the width. Try thickness $= \frac{3}{8}$, then $2{\cdot}5 - (3 \times \frac{3}{8}) = 1{\cdot}375$ and $1\frac{3}{4} \times \frac{3}{4} = 1{\cdot}3125$, $\therefore \frac{3}{8}$ will do, and angle irons will be, say . . . . . $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$

STIFFENERS.

Placed 3 to 6 feet apart according to judgment, say in this case single bent angle irons on each side $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$. One at each end of girder on each side of web closing up end of girder with $\frac{3}{8}$-inch plate, another pair over edge of each abutment, where the shearing stress is at a maximum, and remaining space divided into equal parts, giving even number of stiffeners say 4 feet apart. Length of stiffener over all, approximately = depth of girder + width of flange = 20″ + 12″ = 2′ 8″. The rivets should be continued through web and stiffener as near to the knees as possible, say 4 to 6 inches pitch, or in this case 4 rivets. In larger girders 5″ × $2\frac{1}{2}$″ and 6″ × 3″ tee irons are often used, also double angle irons with gusset plate between.

AREA OF STONE FOR GIRDER BED.

Each bearing surface of girder 15″ × 12″ = 180 sq. inches

$$\frac{180 \times 12 \text{ tons on stone}}{4 \text{ tons on brick}} = 540 \text{ sq. inches area of each stone.}$$

With girders under 50 feet span some engineers give 6 inches outside the girder all round, and over 50 feet span 9 inches, and double these dimensions next to edge of abutment for an "outside" girder.

WIDTH OF STONE.

Three inches chamfer + 1′ 3″ girder +, say 3 inches margin beyond girder = ... 1 foot 9 inches.

LENGTH OF STONE.

Width 1′ 9″ = 21 inches $\frac{540}{21}$ = 25·7 inches; but should be multiple of 4½″ or 9″ to suit bonding of brickwork, make it say = ... 2 feet 3 inches.

THICKNESS OF STONE.

Varies from say ½ width of girder to ½ length of stone. By formula up to 60 feet span, ⅔ span in feet + 4 = thickness in inches, but should be even multiple of 3 inches to suit height of courses in brickwork $\frac{2 \times 20}{5} + 4 = 12$ inches thick. From 60 to 120 feet span, $\frac{1}{180}$ of span + 2 = thickness in feet for sandstone or limestone. Above 120 feet span, the girder should rest on granite.

Size of each stone will therefore be 2′ 3″ × 1′ 9″ × 1′ 0″.

BEDDING OF GIRDER.

Girder to be bedded on two thicknesses of tarred or asphalted roofing felt, with sheet lead between weighing say 8 lbs. per sq. foot. The lead is for equalising the pressure, and the felt to prevent contact between the lead and iron. The lead is now frequently omitted, but in any case the stone bed should be dressed true, allowing for the loss of camber as the load comes on.

FIXING GIRDERS.

When used in a building, wrought-iron girders frequently have cross tails of 3″ × 3″ × ½″ angle iron, 2 to 3 feet long,

bolted across ends on top to hold walls together, but this is objectionable on account of the alteration of length produced by change of temperature. The same objection applies to bolting the ends of girders through a wall. The walls should be entirely self-supporting.

ESTIMATE OF WEIGHT FROM DIMENSIONS.

		lbs.
Top flange, 1st plate . .	$23' 0'' \times 12'' \times \frac{1}{2}''$	= 460
„ 2nd „ . .	say $\frac{2}{3}$ ditto	= 307
Bottom flange, 1st plate .	$23' 0'' \times 12'' \times \frac{1}{2}''$	= 460
„ 2nd „ . .	say $\frac{2}{3}$ ditto	= 307
Angle iron . .	$4 (2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}'') \times 23' 0''$	= 536
Web	$23' 0'' \times 1' 8'' \times \frac{1}{4}''$	= 384
Stiffeners . .	$16 (2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}'') \times 2' 8''$	= 249
End plates . . .	$2 \times 1' 9'' \times 12'' \times \frac{3}{8}''$	= 53
Covers to flanges . .	$(2' 1'' + 1' 6'') \times 12'' \times \frac{1}{2}''$	= 71
„ web . . .	$2 \times 15'' \times 4\frac{1}{2}'' \times \frac{1}{4}''$	= 9
		2836
Rivet heads, say 5 per cent on above . .		= 142
		2978

At 40 lbs. per foot sup. 1 inch thick, the weight = 2978 lbs. = 1·3294 tons, and the approximate weight having been taken at 1·3 tons, the difference is so slight that the designing may be proceeded with; but if any variation is made in the dimensions while designing, care must be taken that the weight allowed for is not materially exceeded, or fresh calculations must be made.

ESTIMATE OF COST, INCLUDING PROFIT.

	£	*s.*	*d.*
Iron at £10 per ton . . .	13	3	1
Labour at £2 10*s.* per ton . .	3	5	9
Erection at £1 per ton . . .	1	6	4
Total cost fixed . .	£17	15	2

Including a priming coat of red lead or oxide paint, but exclusive of any brickwork or masonry, or of any charge for testing. The cost will, of course, vary with the market value of iron and labour. Approximate cost of wrought-iron girder £ = ·036 W*l*. Convenient figures for remembering are wrought iron about 1*d*. per lb., $\frac{3}{4}$ inch rivets 2*d*. each.

QUALITY OF MATERIAL.

The material of this girder is assumed to be of the best quality, and the manufacture carried out under a specification of quality and tests, such as are given in Chapter XII.

Girders are too often designed for present use only, instead of with a view to their stability in twenty or fifty years' time under, perhaps, more severe conditions. Considering the risks of oxidation (particularly in wrought iron), increased loads, settlement of foundations, &c., even the best material and workmanship require a considerable margin of present strength for future safety.

PARABOLA OF STRESS.

For general information on this point see the author's 'Strains in Ironwork.' There are several methods of constructing parabolas; the best for girder work is that shown in Fig. 42. In the development of flange plates and covers (Figs. 44 and 46), if it is desired to take into account any portion of the angle irons, say one side of each, the area must be divided by the width of girder flange to give equivalent value of angle iron in plate thickness. The height of parabola is usually made equal to the calculated thickness of flanges, e. g. in this girder top flange = ·9375, bottom flange = 1 inch. When the height of a parabola does not exceed $\frac{1}{8}$ of its base, it may be replaced without sensible error by a circular arc, with the centre upon a prolongation of the axis of the parabola. In practical work this method would usually require trammels.

DESIGN.

In working out the design, all the preceding dimensions are subject to modification if required; but a good draughtsman makes very little deviation from his first figures. The accompanying sheet (Plate 4) explains itself.

CHAPTER V.

ROLLED JOIST.

(See Plate 5.)

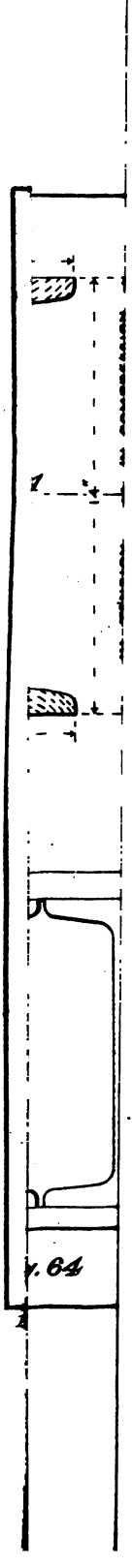
REQUIRED a rolled iron girder, 20 feet span, supported at the ends, to carry one rod reduced of 9-inch brickwork.

Rolled girders or joists are manufactured from puddled wrought-iron bars, cut up into short lengths, piled, heated, welded under steam hammer, rolled into a *bloom*, re-heated, passed through grooved rollers approximating to, and finally exactly of, the finished section required. The iron is of common quality, and generally red-short, i. e. containing sulphur, and therefore unsuitable for working at a red heat.

Small rolled joists are much used for fire-proof floors, being bedded in concrete. They are protected against rapid heating in case of fire by the concrete enveloping them, but they are not strengthened by it unless they are so narrow as to be deficient in lateral stiffness without some such support. The strength of the concrete itself as a beam should be neglected, unless the span is so short as to render it appreciable, say thickness not less than $\frac{1}{8}$ span.

Larger rolled joists are used in place of cast girders or plate girders, especially by architects, and are in many cases very suitable, but there are limits to their economical use which are not always observed.

Generally, a section can be selected from a maker's list,


v. 64

where the various sizes are tabulated with the safe loads for different spans. We will, nevertheless, go through the calculations first, and afterwards compare the result with the lists.

A ROD OF REDUCED BRICKWORK contains 272 feet 3 inches super, always called 272 feet of 1½ bricks thick, or 306 cubic feet, and when laid in mortar weighs,

say . . . . . 15 tons.

The 306 cubic feet is made up of 71 cubic feet of mortar and 235 cubic feet of bricks, as follows :—

APPROXIMATE CONTENTS AND COST OF A ROD OF BRICKWORK.

	£	s.	d.
1 cubic yard ground stone lime at 16*s.* .	0	16	0
3½ loads clean sharp sand at 5*s.* . .	0	17	6
Add labour, say	0	7	6
„ water „	0	5	0
	2	6	0
Forming 71 cubic feet, or say 3 yards mortar at 15*s.* 4*d.* per yard . .	2	6	0
4350 hard well-burnt London stocks (containing 235 cubic feet net) at 35*s.* per thousand	7	12	3
Add labour, say . .	3	0	0
Scaffolding, say	0	10	0
Total net cost per rod .	13	8	3
Add profit, 15 per cent. .	2	0	3
	15	8	6

Say £15 10*s.* per rod.

Taking finished work roughly at 1*d.* per brick = 4350 at 1*d.* = £18 2*s.* 6*d.* per rod.

Builders usually reckon only 4300 bricks in a rod of brickwork, on account of voids in the walling.

HEIGHT OF WALL.

If one brick thick, = 9 inches, it will be

$$\frac{306 \text{ cub. ft. in rod}}{\cdot 75 \text{ ft. thickness} \times 20 \text{ ft. span}} = 20\cdot 4, \text{ say} \quad . \quad 20 \text{ ft. } 4\tfrac{3}{4} \text{ in.}$$

If $1\frac{1}{2}$ bricks thick, = $13\frac{1}{2}$ inches, commonly called 14 inches, it will be

$$\frac{306}{1\cdot 125 \times 20} = 13\cdot 6, \text{ say} \quad . \quad . \quad . \quad 13 \text{ feet } 7\tfrac{1}{4} \text{ inches.}$$

But usually in London work the height will be a multiple of 3 inches as four courses generally occupy 1 foot in height. The rougher the bricks the thicker the requisite joint to give a uniform bed. The rule for best work is that no four courses of bricks, including three joints, shall gauge more than one inch beyond the same bricks laid dry.

If the brickwork were thicker than 9 inches, it would be carried by a wrought-iron plate riveted on top of rolled joist, or by pieces of 2-inch York paving laid on the joist, equal in width to the thickness of wall.

CLEAR SPAN,

say . . . . . 20 feet.

DEPTH OF GIRDER.

This is usually measured over all, including both flanges, and may vary from 3 inches to about 15 inches, or more in

special cases by rolling top and bottom half separately, and welding the web. The depth should be from $\frac{1}{15}$ to $\frac{1}{20}$ of the span, but is sometimes very shallow on account of the limited space available for thickness of floor, say $\frac{1}{30}$ to $\frac{1}{40}$ of span in extreme cases where deflection is of no consequence.

In this case $\frac{1}{15}$ to $\frac{1}{20}$ = 16 inches to 12 inches, make it, say . . . . . 14 inches.

The strength varies directly as the weight of the flanges for any given depth, but a deep girder will be stronger than a shallow girder of the same weight. The weight apart from the dimensions is no criterion of the strength.

WIDTH OF FLANGE.

May be of from $\frac{1}{3}$ depth of girder in large sections to full depth of girder in small sections, generally $\frac{1}{3}$ to $\frac{2}{3}$; an average may be taken of $\frac{1}{2}$ depth, but the width should not be less than $\frac{1}{40}$ of the span, when unsupported at the sides, in order to guard against the risk of lateral flexure of the compression flange.

$\frac{1}{3}$ 14 inches = $4\frac{2}{3}$ inches, $\frac{2}{3}$ 14 inches = $9\frac{1}{3}$ inches, $\frac{1}{2}$ 14 inches = 7 inches, $\frac{20 \times 12}{40}$ = 6 inches.

∴ make this, say. . . . . 6 inches.

MARKET DESCRIPTION.

The market description of rolled joists and girders is by the weight in lbs. per foot run for a given depth and breadth. They can be rolled thicker, say $\frac{1}{4}$ inch as maximum, or up to 10 per cent. heavier, to increase their strength. The same rolls being used the difference is made in the thickness of web

and width of flanges. The variation of thickness in rolling is not a good method of adjusting their strength, as any departure from the normal proportion will make the web too thick or too thin for economical use of the weight.

When the flanges have much taper the advantage of thicker rolling is increased.

WEIGHT OF GIRDER.

The actual weight depends upon the proportions adopted for the section, approximately it will be about

$$\frac{W\,L}{D} \times c = \text{lbs. per foot run,}$$

where W = safe distributed load in tons.
L = span in feet.
D = depth in feet.
c = constant = ·2 to ·25.

$\frac{15 \times 20}{1 \cdot 166} \times \cdot 2 = 51 \cdot 4$ lbs., $\frac{15 \times 20}{1 \cdot 166} \times \cdot 25 = 64 \cdot 32$ lbs.

say . . . . . 60 lbs. per foot run.

LOAD TO EDGE OF BEARING SURFACES.

$15 \text{ tons} + 20\left(\frac{60}{2240}\right) =$ . . . . 15·536 tons.

LENGTH OVER BEARING SURFACES.

Say sandstone template on stock brickwork in cement, the safe loads being respectively 12 and 6 tons per foot super. (See p. 43.)

When the load to be carried terminates at edge of bearing surfaces, the length of each bearing surface in feet will be

$$= \frac{\frac{1}{2}\text{ (external load + weight of girder)}}{\text{safe load on bed} \times \text{width bearing surface feet}}$$

$$= \frac{\frac{1}{2}(15 + \cdot 536)}{12 \times \cdot 5} = 1 \cdot 29 \text{ feet};$$

but in this case the brickwork will, without a doubt, be continued over the girder as far as the latter extends, and therefore the bearing surface will have this additional load to carry. Then we must find the equivalent per foot run, thus

$$\frac{15 \cdot 536}{20 \text{ feet span}} = \cdot 7768 \text{ tons per foot run,}$$

and $$l = \frac{W}{s\,b - w}, \text{ as at p. 44,}$$

or $$l = \frac{15 \cdot 536}{(12 \times \frac{6}{12}) - \cdot 7768} = 2 \cdot 97, \text{ or say 3 feet total,}$$

= say . . . . . 18 inches each end,

being $\frac{1}{2}$ (2·97) − 1·29 = ·195 feet, or $2\frac{3}{8}$ inches more than common calculation would show to be necessary.

CHAMFER ON STONE.

As the chamfer to prevent chipping at the edge of bearing surface adds to the span, it is desirable when possible to let the stone beds project by the amount of the chamfer to leave the span unaltered, see Fig. 52.

$$\frac{W \text{ tons} + \sqrt{\text{span feet}}}{10} = \frac{15 + \sqrt{20}}{10} = 1 \cdot 947, \text{ say 2 inches.}$$

In architectural work, for dead loads only, this may be reduced by one-half, making it, say . . . . 1 inch.

It is important not to omit the chamfer on stone under rolled joists owing to the absence of camber, and subsequent deflection. When bedded on tarred felt the risk of fracture is minimised but not entirely removed.

FACTOR OF SAFETY.

The safe loads specified by makers are intended to be, in most cases, not greater than $\frac{1}{4}$ breaking weight, and in some cases much less, in order that the deflection may not be objectionable. The Butterley Iron Company give $\frac{1}{3}$ breaking weight as the safe steady load, and $\frac{1}{4}$ to $\frac{1}{5}$ as safe moving load.

For a live load, where the depth is less than $\frac{1}{15}$ of the span, a factor of safety of $\frac{1}{6}$ should be taken.

In ordinary wrought-iron plate girders 5 tons in tension and 4 tons in compression are considered to be the proper working loads per sq. inch; rolled joists, being of harder iron, may well be allowed 5 tons per sq. inch for ordinary cases, both in tension and compression, the flanges being made equal, and any risk from reversal of position avoided.

AREA OF FLANGES.

By ordinary formula for rolled joists, supported at both ends, loaded in the middle,

$$W = C \frac{a\,d}{L}$$

where W = breaking weight tons centre;

a = area bottom flange + $\frac{1}{6}$ depth web (remainder of web assumed to resist shearing only);

d = depth in inches;

L = length or span in feet;

C = constant = 7 for rolled iron joists;

or by transposition,

$$a = \frac{W L}{C d};$$

but the load being distributed, only half the stress will be induced, and therefore, allowing a factor of safety of $\frac{1}{4}$,

$$a = \frac{15{\cdot}536}{2} \times 4 \times \frac{20}{7 \times 14} = 6{\cdot}34.$$

$\therefore$ each = ... 6·34 sq. inches.

MEAN THICKNESS OF FLANGES

should be from $\frac{1}{6}$ to $\frac{1}{12}$ of the width, width is 6 inches, therefore thickness 1 inch to $\frac{1}{2}$ inch,

mean thickness, say . . . . . $\frac{3}{4}$ inch,

but area of flange + $\frac{1}{6}$ area of web, must = 6·4 inches, therefore, actual thickness is subject to confirmation.

THICKNESS OF WEB

should be from $\frac{1}{16}$ to $\frac{1}{24}$ depth, which is very much in excess of the web in plate girders, owing to the absence of vertical angle iron or other stiffeners,

$\frac{14}{16} = {\cdot}875$, $\frac{14}{24} = {\cdot}583$, say . . . . $\frac{5}{8}$ inch.

SECTION OF GIRDER.

This may now be sketched out as in Fig. 53, and the detail of flange sketched out for calculation, as in Fig. 54.

SECTION OF FLANGES.

Assume dimensions that will give about the area required, and be in accordance with the foregoing proportions. The

first trial will probably show some modification is necessary to give the precise area required, and ultimately Fig. 55 will be obtained.

DETAILED AREA OF FLANGE.

This may be measured from drawing, by planimeter, or calculated in detail as follows:—

$a + a = \cdot375 \times \cdot25 \times 2$	$= \cdot1875$		
$b + b = \frac{\cdot1963}{4} \times 2$	$= \cdot0982$	Flange	
$c + c = (\cdot625 + 1) \times 1\cdot9375$	$= 3\cdot1484$	$5\cdot0591$	
$d = 1\cdot625 \times 1$	$= 1\cdot6250$		
$e + e = (1 \times \cdot5) - \frac{\cdot7854}{2}$	$= \cdot1073$	Web	
$f = 2 \times \cdot625$	$= 1\cdot2500$	$1\cdot3573$	
	$6\cdot4164$ sq. inches		

or very slightly in excess of area required by formula.

ACTUAL WEIGHT OF GIRDER.

Area of flange + $\frac{1}{6}$ web as above	=	$6\cdot4164$
do. do.	=	$6\cdot4164$
Remainder of web, $8 \times \cdot625$	=	$5\cdot0000$
Total sq. inches area of section	=	$17\cdot8328$
		$3\frac{1}{3}$
		$53\cdot4984$
		$5\cdot9443$
		$59\cdot4427$

say . . . . . 60 lbs. per foot run.

COMPARISON OF AGENTS' LISTS.

Rolled joist 14″ × 6″ × 20 feet span.

	per foot run.	
The Butterley Co. (14 × 6¼)	59 to 62 lbs.	15·4 tons safe load
W. H. Lindsay	60 „	16 „
Homan & Rodgers	60 „	15·4 „
Measures Bros.	60 „	15·2 „
M. T. Shaw & Co.	60 „	15·0 „
J. O. Gardner & Co.	60 „	15·0 „
Macnaught, Robertson & Co.	60 „	14·8 „
Drew, Bear, Ransome & Perks	58 „	12·5 „
Rownson, Drew & Co... ..	56 „	11·9 „
W. Rammage & Co.	56 „	15·0 „
C. Williams & Co. (14 × 5½)	56 „	13·5 „

In all cases, span × safe load = constant, except for extreme spans, so that the constant for any girder divided by the span in feet = safe load in tons distributed. See table at p. 82.

The variation in safe load given above may be accounted for by slight differences in the proportionate thicknesses of the parts; it will, however, be instructive to ascertain by other formulæ the strength of the rolled joist we have decided upon, the full section of which is given in Fig. 56.

STRENGTH BY LEVERAGE.

This method is simple, and easy to understand on the supposition that the flanges resist the whole of the bending stresses. See p. 35, 'Strains in Ironwork.'

$$\text{Area 1 flange} = \frac{\frac{1}{4}\text{ distributed load tons} \times \frac{1}{2}\text{ span feet}}{\text{mean depth ft.} \times 5\text{ tons per sq. in. safe load}},$$

or
$$a = \frac{\frac{1}{4}W \times \frac{1}{2}l}{d \times 5},$$

and by transposition $W = \frac{40\,a\,d}{l}$.

$$d = \text{mean depth feet} = \frac{14 - \cdot 875}{12} = 1\cdot 094,$$

$$\therefore\ W = \frac{40 \times 5\cdot 06 \times 1\cdot 094}{20} = 11\cdot 07 \text{ tons.}$$

In case we should have made some mistake in our leverages, let us try the ordinary formula for a flanged beam, viz.:—

$$\therefore\ \text{Stress in flange} = \frac{W\,l}{8\,d},$$

and allowing 5 tons per square inch safe load,

$$\text{Area of flange} = \frac{W\,l}{8\,d} \div 5,$$

and by transposition,

$$W = \frac{40\,a\,d}{l} \text{ as before.}$$

We know the formula to be approximately true for large girders, and we see that it is founded upon a consideration of the leverages involved, but a rolled joist being homogeneous, every fibre connected to every adjacent fibre, and having a web of abnormal thickness, we should be justified in taking more than the flange only as resisting the bending stresses; and as the ultimate strength of the iron itself is usually 21 tons per sq. inch, the safe load will be higher than we have taken. Substituting these values for those previously taken, we shall have

$\frac{21}{4} = 5{\cdot}25$ tons per sq. inch safe load, $5{\cdot}25 \times 8 = 42$, and

$$W = \frac{42\,a\,d}{l} = \frac{42 \times 6{\cdot}4 \times 1{\cdot}094}{20} = \quad . \quad 14{\cdot}7 \text{ tons.}$$

STRENGTH BY BOX'S SPECIAL RULE.

Box* considers that wrought-iron beams always fail on the compression side, and cast-iron on the tension side; he therefore assumes the neutral axis of a rolled joist supported at the ends and loaded on the top to be coincident with the lower surface of the beam, the whole sectional area being in compression in the ratio of its distance from the neutral axis. He compares the results of calculation on this supposition with actual experiments and shows a very close approximation. In Fig. 57 N A will be the neutral axis, and the dimensions required are marked upon the section.

Width top flange × diff. between sqs. of dist. from N A
$= 6\,(14^2 - 13{\cdot}125^2) = 142{\cdot}41.$

Thickness web × diff. between sqs. of dist. from N A
$= {\cdot}625\,(13{\cdot}125^2 - {\cdot}875^2) = 107{\cdot}19.$

Width bottom flange × diff. between sqs. of dist. from N A
$= 6\,({\cdot}875^2 - 0^2) = 4{\cdot}59,$
then $142{\cdot}41 + 107{\cdot}19 + 4{\cdot}59 = 254{\cdot}19.$

Coefficient of transverse strength or breaking weight in lbs. on centre of unit beam = 3200, and $\frac{254{\cdot}19 \times 3200}{20 \text{ feet span}} = 40670$ lbs. breaking weight centre; for load distributed, × 2; for safe load (Box $\frac{1}{3}$ breaking weight) ÷ 3; for load in tons, ÷ 2240,

then $40{,}670 \times \frac{2}{3 \times 2240} = \quad . \quad . \quad . \quad 12{\cdot}104$ tons.

* 'Strength of Materials,' p. 195 and p. 213.

STRENGTH BY IVES' METHOD.

In a back number of the 'Foreman Engineer and Draughtsman' (p. 141, August 1876) some calculations of rolled joists are given upon the following method:

Neutral axis through centre of gravity of section = centre of depth, see Fig. 58.

f = area of bottom flange;
m = mean thickness ditto;
w = area of web;
d = depth of web;
s = total sectional area of joist;
g = distance of centre of gravity of each half of section from outer edge;

$$g = \frac{(f \times \frac{1}{2}m) + \frac{1}{2}w(\frac{1}{4}d + m)}{\frac{1}{2}s} = \frac{fm + w(\frac{1}{4}d + m)}{s}$$

$$= \frac{(5{\cdot}06 \times {\cdot}875) + 7{\cdot}715(\frac{12{\cdot}25}{4} + {\cdot}875)}{17{\cdot}83} = \frac{4{\cdot}4275 + 30{\cdot}3778}{17{\cdot}83} = 1{\cdot}95.$$

e = effective depth of joist
= total depth $- 2g = 14 - 2(1{\cdot}95) = 10{\cdot}1$;
t = ultimate tensile strength, tons per square inch;
l = clear span in inches.

$$\text{B. W. tons distributed} = \frac{8 \times \frac{1}{2}s \times t \times e}{l}$$

$$= \frac{8 \times 8{\cdot}92 \times 21 \times 10{\cdot}1}{20 \times 12} = 63{\cdot}064.$$

Safe load at $\frac{1}{4}$ breaking weight $= \frac{63{\cdot}064}{4} =$. 15·766 tons.

STRENGTH BY D. K. CLARK'S FORMULA.

(Law's 'Civil Engineering,' p. 13.)

a = area of one flange + ¼ area of web calculated on total depth;

d = depth in inches;

S = span in feet;

c = constant = 133;

W = breaking weight in cwts. in centre.

Then, $$W = \frac{133\,a\,d}{S}$$

$$= \frac{133 \times (5{\cdot}059 + 2{\cdot}188) \times 14}{20} = 674{\cdot}7 \text{ cwt.}$$

÷ 20 for tons, × 2 for distributed load, ÷ 4 for safe load,

$$\therefore \frac{674{\cdot}7 \times 2}{20 \times 4} \quad . \quad . \quad . \quad . \quad . \quad . \quad 16{\cdot}87 \text{ tons.}$$

STRENGTH BY WINTON'S RULE.

('Modern Workshop Practice,' p. 199.)

$$\text{B.W. tons centre} = \frac{\text{area bott. fl.} \times \text{total depth inches}}{\text{span inches}} \times 80$$

$$\times \frac{2 \text{ for distributed load}}{4 \text{ for safe load}}$$

$$= \frac{5{\cdot}06 \times 14}{20 \times 12} \times 80 \times \tfrac{2}{4} = . \quad . \quad . \quad . \quad 11{\cdot}807 \text{ tons.}$$

STRENGTH BY MOLESWORTH.

At p. 123, 'Molesworth's Pocket Book,' a method is given for calculating the strength by what is supposed to be the moment of inertia.

B.W. cwts. $= \frac{8 \text{ K B D}^2}{\text{L}}$ for ordinary beams, and substituting the value of V for B D^2, and making b to correspond with Fig. 59,

B.W. cwts. $= \frac{8 \text{ K (B D}^2 - b d^2)}{\text{L}}$ whence it will be seen that in place of the B D^2 for ordinary beams, we get the difference between the B D^2 of the containing rectangle, Fig. 60, and the $b d^2$ of the deficient rectangle (see Fig. 61); but $k = 68$ for wrought iron, safe load $= \frac{1}{4}$, 20 cwt. $=$ 1 ton, $\therefore$ safe load tons distributed

$$= \frac{8 \times 68 \times (\text{B D}^2 - b d^2)}{\text{L} \times 20 \times 4} = 6{\cdot}8 \frac{\text{B D}^2 - b d^2}{\text{L}}$$

$$= 6{\cdot}8 \frac{1176 - 806{\cdot}58}{20 \times 12} = \quad . \quad . \quad . \quad . \quad 10{\cdot}46 \text{ tons.}$$

This result is the worst so far, and well illustrates the inaccuracy of the method. This is due to the true moment of inertia not being employed, although it is given on the same page of Molesworth (I =). The reason of the inaccuracy of the formula (V =) is that when the beam is breaking with the deflection due to section A, Fig. 60, section B is only under a deflection equal to half that at its breaking weight, and therefore only half its strength should be subtracted from the full strength of A.

We will now check the results by a more precise method,

viz. by using the modulus of section founded upon the true moment of inertia. For exact calculation the areas of infinitely small portions must be taken, and therefore recourse must generally be had to formulæ such as the following :—

MOMENT OF INERTIA.

The moment of inertia (I) of a section is the summation of the areas of all its individual parts multiplied by the squares of their distances from the neutral axis, thus $I = \Sigma a y^2$, or for a beam with equal flanges, $I = \frac{B D^3 - b d^3}{12}$, see Fig. 59.

MODULUS OF SECTION.

The modulus of section (Z), or strength modulus, is proportional to the moment of resistance of the section. It is the moment of inertia divided by the distance from the neutral axis to the furthest part on the extended or compressed side.

$Z = \frac{I}{y}$ ∴ for a beam with equal flanges, $Z = \frac{B D^3 - b d^3}{6 D}$.

MODULUS OF RUPTURE.

The theoretical value of the modulus of rupture (C) for transverse strain is the resistance of the material to direct tension or compression ;* but it is found from experiments on cross-breaking that this value is from various causes not sufficiently high, and Prof. Rankine adopted a modulus which is 18 times the load required to break a bar of 1 sq. inch in

* Humber's 'Handy Book on Strains,' p. 62.

section, supported on two points 1 foot apart, and loaded in the middle between the supports, viz.:

C = wrought iron 42,000 lbs., cast iron 40,000 lbs., timber average 7500 lbs.

According to the experiments tabulated by Box,* a higher value than this would be proper for wrought iron and lower for cast. His notes would give the modulus of rupture for flanged beams of wrought iron as C = 57,600 lbs.

As there is considerable difference between these figures we will take a mean, or say . . . . 50,000 lbs.

MOMENT OF RESISTANCE.

The moment of resistance (R) of a section is the modulus of the section × the modulus of rupture, R = Z C.

BENDING MOMENT.

The bending moment (M), known also as the moment of flexure, moment of rupture, or leverage effect of load, is the total load (= W in lbs.) multiplied by the span (= l in inches) and divided by the coefficient of reaction (see p. 7), $M = \frac{W l}{k}$.

STRENGTH BY MODULUS OF SECTION.

$$\text{Effort} = \text{Resistance},$$

$$M = R,$$

$$\frac{W l}{k} = Z C,$$

* 'Strength of Materials,' p. 192.

$$\therefore W = Z C \frac{k}{l}$$

$$= \frac{B D^3 - b d^3}{6 D} \times 50{,}000 \times \frac{8}{l}$$

$$= \frac{(6 \times 14^3) - (5{\cdot}375 \times 12{\cdot}25^3)}{6 \times 14} \times 50{,}000 \times \frac{8}{20 \times 12}$$

$$= \frac{16{,}464 - 9880}{84} \times \frac{50{,}000 \times 8}{20 \times 12}$$

$$= 78{\cdot}38 \times 1667 = 130{,}659{\cdot}5 \text{ lbs.}$$

which at 2240 lbs. = 1 ton, and factor of safety $\frac{1}{4}$,

$$= \frac{130{,}659{\cdot}5}{2240 \times 4} = \quad . \quad . \quad . \quad . \quad . \quad 14{\cdot}58 \text{ tons.}$$

This method is theoretically true when the resistance to tension and compression are equal. The accuracy of the result depends upon the approximation of the inner breadth and depth (b, d) to the true mean.

COMPARISON OF RESULTS.

By ordinary formula	. .	15·536 tons.
„ leverage	. . .	14·7 „
„ Box's special rule	. .	12·104 „
„ Ives' method	. . .	15·766 „
„ D. K. Clark's formula	.	16·87 „
„ Winton's rule	. . .	11·807 „
„ Molesworth, p. 123	. .	10·46 „
„ modulus of section	. .	14·58 „

The latter is the most reliable method, and we may therefore rest satisfied that the section we have adopted will be about right.

No notice has been taken of the reduction of stress consequent upon building the ends of the girder into the brickwork ; this may be looked upon as simply saving deflection, as rolled joists are seldom cambered.

TEST LOADS.

Measures Bros. state on their section sheet that, "Every section of joists has been loaded by dead weight to above the indicated load without any material deflection, and in no case where they have been loaded to four times these indicated safe loads has the joist broken or failed to sustain the weight."

Drew, Bear, Ransome & Co. state that their joists are calculated for a maximum strain of 5 tons per sq. inch.

Moser & Sons state, "Experiments have been made upon two girders of each size, placed upon bearings twice the distance apart in feet that the section is high in inches, the girders being about 12 feet from each other, and loaded with bar iron placed in the centre."

A load "in centre" should not have a bearing wider than $\frac{1}{80}$ of the span, or it will not fairly represent a central load as calculated for; and if the bearing be narrower than $\frac{1}{100}$ span, it may present difficulty in its application, and also be liable to cut the flange.

TOTAL LENGTH OF GIRDER.

Clear span + 2 bearing surfaces = 20 + 3 = . 23 feet.
Stock lengths usually vary from 10 feet up to 30 feet, rising by 1 foot with some makers, and 2 feet with others.

TOTAL WEIGHT.

23 feet × 60 lbs. per foot ÷ 2240 = . . ·616 of ton.

DEFLECTION.

$$\text{Approximately} = \frac{\cdot 012 \times \text{span feet}^2}{\text{mean depth inches}}$$

$$= \frac{\cdot 012 \times 20^2}{12 \cdot 25} \quad . \quad . \quad . \quad . \quad . \quad \cdot 392 \text{ inches.}$$

$$\text{More accurately} = \frac{L^3 W c}{D^3 B - d^3 b}$$

where L = length in feet, W = load distributed in tons, c = ·018, D = total depth inches, B = total breadth inches, d = depth inches between flanges, b = breadth flange inches minus web.

Then

$$\frac{20^3 \times 15 \cdot 616 \times \cdot 018}{(14^3 \times 6) - (12 \cdot 25^3 \times 5 \cdot 375)} = \cdot 356, \text{ say } . \quad . \quad \tfrac{3}{8} \text{ inch.}$$

The deflection of rolled joists used in floors may be as much as $\frac{1}{40}$ inch per foot, or $\frac{1}{60}$ inch when supporting concrete arches, but for an external wall the deflection should not exceed $\frac{1}{100}$ inch per foot span. In this case the girder being built in 18 inches at each end, the actual deflection will probably be within this amount.

COST OF GIRDER.

·616 ton at £8 per ton = £4 18*s.* 6*d.*, delivered on the works, exclusive of any of the extras detailed below.

CHECKING WEIGHT.

The invoice weight should be tested and checked with the weights given on section sheets, as the actual weights are sometimes lower, and the strength is, in such cases, lower than estimated for.

GENERAL NOTES ON COST.

English rolled joists cost about the same as B.B. Staff. bar iron, say £8 per ton. Belgian rolled joists are about 25 per cent. cheaper, say £6 per ton, but not so reliable.

The ordinary quality has a tensile strength of 21 tons per sq. inch of section with the grain.

Sawing ends square to required length while hot is included in the price. A cutting margin of 1 inch under or over specification is claimed as fulfilling this condition.

Cutting to "exact length," i. e. ¼ inch under or over specified length, is charged 3*s.* per ton extra.

Cutting cold to "dead length," or perfectly true, 5*s.* to 7*s.* 6*d.* per ton extra.

Joists or girders above 30 feet in length, 1*s.* 6*d.* per ton per foot extra.

For quantities under 5 tons, and for delivery within 3 weeks, or as rolls next go in, 5*s.* per ton extra.

For delivery from stock promptly, for quantities above 5 tons, 10*s.* per ton extra.

For delivery from stock promptly, for quantities below 5 tons, 15*s.* per ton extra.

When joists are required to be cut down from stock, the

whole joist is charged, and 1*s.* 6*d.* per cut, the scrap being credited at 40*s.* per ton.

Round holes in flanges 2*d.*, webs 1*d.* each.

Oval holes in flanges 3*d.*, webs 2*d.* each.

Cold straightening when required is charged as an extra.

Special quotations can be obtained for girders of the best iron or mild steel.

COMBINATIONS OF ROLLED JOISTS.

In order to meet the case of heavy loads, various combinations of rolled joists are made, formerly called "patent girders," now generally "compound girders." Some of these are shown in Figs. 62 to 67.

As a rule they are cheap and useful, but in many of them the metal is not disposed in the most economical manner as regards the weight compared with the strength.

The strength of a compound girder made of two similar rolled joists, Fig. 63, is practically equal to twice the strength of one of the joists composing it; and one as Figs. 66 or 67 is equal to the combined strength of the component joists, but they must be riveted together for not less than $\frac{1}{4}$ of their length from each end to resist the horizontal shear stress.

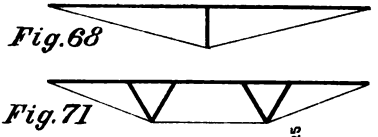
TABLE OF ROLLED JOISTS.

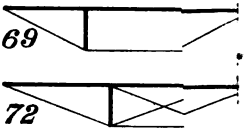
The following table gives the constants for all the sections in Measures Bros'. list. To select a section, multiply the required span by the distributed load in tons, which will give the constant. Thus 20 × 15 = 300, say 305·2, or No. 35 L. 14″ × 6″.

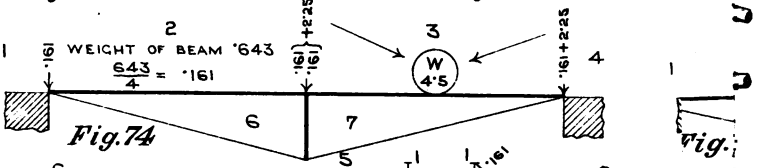
No. of Section.	Weight in lbs. per foot run.	Depth and Breadth.	Limits of Span.	Constant.
		inches.	feet.	
010	7	4 × 1¾	6 to 14	10·9
012	9	4¾ × 1¾	6 „ 14	13·5
* 8 L.	10	3 × 3	6 „ 10	11·2
016	11	6¼ × 2	6 „ 18	24·4
* 10	12	4 × 3	6 „ 14	18·9
* 12	13	4¾ × 3	6 „ 18	25·5
018	14	7⅛ × 2¼	6 „ 18	32·8
20	15	8 × 2½	7 „ 26	48·6
* 16	16	6¼ × 3⅛	8 „ 20	38·3
* 18	20	7 × 3¾	8 „ 24	55·7
* 20	22	8 × 4	10 „ 24	77·4
*145	23	5 × 4½	6 „ 18	44·8
* 23	24	9¼ × 3¾	10 „ 22	82·1
*153	29	6 × 5	6 „ 23	68·8
* 20 L. H.	29	8 × 5	8 „ 28	92·1
* 24 L. H.	29	9½ × 4½	10 „ 24	115·5
* 25 L.	32	10 × 4½	10 „ 26	129·8
20 H. H.	36	8 × 6	8 „ 28	112·0
* 25 L. H.	36	10 × 5	10 „ 26	147·9
* 30 L.	42	12 × 5	10 „ 26	217·7
25 H. H.	56	10 × 6	10 „ 32	188·6
* 30 M.	56	12 × 6	10 „ 34	267·2
* 35 L.	60	14 × 6	12 „ 34	305·2
* 40 L.	62	16 × 6	16 „ 34	370·0
45 L.	82	18 × 7 & 17¾ × 6¾	10 „ 40	400·0
50 L.	100	19¾ × 7⅛ & 20 × 8	10 „ 40	500·0

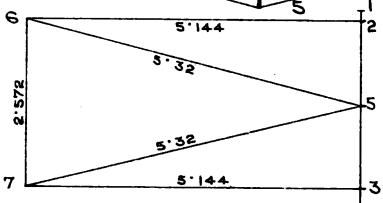
Those marked * are kept in stock, remainder rolled to order.

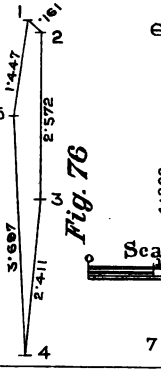
TRUSSED BEAM

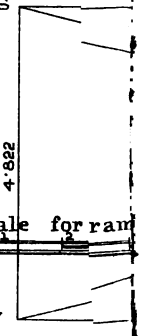


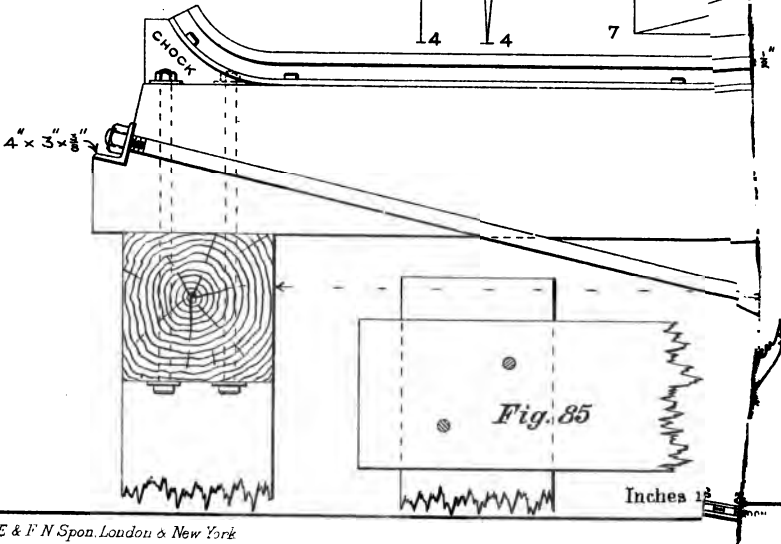












E & F N Spon, London & New York

CHAPTER VI.

TRUSSED BEAM.

(See Plate 6.)

TRUSSED FIR BEAM 20 FEET SPAN SUPPORTED AT THE ENDS TO CARRY A ROLLING LOAD OF $4\frac{1}{2}$ TONS.

TRUSSED beams are used for overhead travelling cranes and gantries; for gangways between warehouses; for road and railway bridges of short span where first cost is an object, as in temporary structures; for carrying a load such as a roof over a wide opening; for long purlins, &c.

The trussing may be either external, with tension rods, or internal, with compression bars; the latter form is now obsolete.

Whenever the depth will allow of external trussing with tie rods and struts, one of the arrangements shown in Figs. 68 to 72 will be found suitable for small spans.

The deeper the struts the smaller the stresses, and therefore the lighter the whole arrangement.

With a rolling load Fig. 68 or Fig. 72 will be the only forms suitable. With two struts as in Fig. 69, when the load is over one of them, as B Fig. 73, the stress in the tie rod will tend to straighten it between the points B C D and therefore to lift strut C and with it the beam. For this reason the counterbracing should be inserted as shown in Fig. 72, whenever more than one strut is used to carry a rolling load.

A rolling load is generally distributed over four wheels, say two on each beam, but as the description gives no particulars of the centre distance between wheels we will assume the load concentrated upon one wheel, and the beam will then be actually a little stronger than our calculations will show; or more correctly, the stresses will be less than we are providing for.

OUTLINE OF TRUSS.

Before we can decide whether to adopt Fig. 68 or Fig. 72 we must know approximately what the stresses will be. Evidently the weak part of Fig. 68 will be midway between the centre and one end, where the beam will be under the maximum transverse stress. To prove this we must know the size and quality of the timber and the compression it will have to resist.

TIMBER IN BALK.

That known as fir timber or Baltic fir is the product of the *Pinus sylvestris*, Northern pine or Scotch fir, and comes principally from the ports in the Baltic, viz.:—

Stettin (Prussian)	.	18 to 20 inches square.
Dantzic „	.	13 „ 16 „
Memel „	.	12 „ 14 „
Riga (Russian)	.	10 „ 12 „
Swedish . .	.	10 „ 12 „
Norwegian .	.	8 „ 9 „

The difference in size depends upon the soil and other circumstances. The length varies from 25 to 45 feet. Dantzic

is the longest, and is generally coarse but strong, with a number of large and often dead knots, top ends most frequently knotty. Memel is very uniform and durable, sometimes a little rounding on the face, for most purposes a very convenient size. Riga is usually the best in quality and most uniform, being straight-grained with small hard knots, but does not exceed 12″ × 12″ and is not quite so stiff as the others.

The best trees are imported round for masts, the next best sawn into planks, deals and battens, and the remainder rough hewn into square balks. The square balks are divided according to quality into crown, best middling, good middling and common middling. Crown is often specified but seldom seen, good middling is sufficient for ordinary purposes and mostly used. The quality is shown by scribe marks on one of the sides. The best quality has fewest marks. Good middlings are marked as follows:

Stettin, at end, two strokes and line across.
Dantzic, in centre, stroke with two crosses on it.
Memel, at end, three strokes.
Riga, in centre, two strokes crossing two others.

There are many other marks signifying the name of importer, the number of the raft, number of balk, cubic contents, &c. They are most numerous on Dantzic timber.

Pitch pine timber can be obtained 16 inches square and 60 feet long. Although very strong when new, it becomes very brittle when the resin has dried out and is not safe for carrying loads. For further information on timber see the author's 'Joints in Woodwork,' and also 'Timber Piling.'

CLEAR SPAN

say . . . . . 20 feet.

DEPTH OF TRUSSING.

Usually $\frac{1}{8}$ of the span, measuring from centre of beam to centre of tie rod below.

$\frac{20}{8} = 2 \cdot 5$, say . . . . . 2 feet 6 inches.

APPROXIMATE WEIGHT OF TRUSSED BEAM.

W = external load in tons.

L = span in feet.

Approximate weight in tons $= \frac{W L}{270}$ for distributed load, or $\frac{W L}{140}$ for rolling load; the constant being taken from previous notes of actual weights.

$\therefore \frac{W L}{140} = \frac{4 \cdot 5 \times 20}{140} =$ . . . . . ·643 of a ton,

exclusive of any rail or other accessory. This may be taken as half acting in the middle of span and half resting on supports.

For a rolling load a rail will be required on top; this need not enter into the calculation of the stresses, as the extra depth and stiffness it will give to the beam will fully compensate for its weight.

SCANTLING REQUIRED TO SUPPORT LOAD BETWEEN STRUT AND ONE END.

Ordinary formula for rectangular beam

$$\text{B.W. cwts. centre} = C\frac{b\,d^2}{L}.$$

Assume beam Memel fir, square section, factor of safety 10, C = 4, and W = safe load tons centre. L = span in feet; b = breadth of side in inches;
then b and d being equal,

$$b = \sqrt[3]{50\,W\,L}$$

$$= \sqrt[3]{50 \times 4{\cdot}5 \times 10} = \quad . \quad : \quad . \quad 13{\cdot}1 \text{ inches square.}$$

TRUSS STRESSES WHEN LOAD IS HALF-WAY BETWEEN STRUT AND ONE END.

Draw truss, number the external and internal spaces and apportion the loads as Fig. 74.

Draw reciprocal diagram as Fig. 75, and measure off the stresses. (For full explanation see the author's 'Strains in Ironwork.') Compression in 7-3 = 5·144 tons. This may also be seen by inspection, viz. strut = $\frac{1}{4}$ of $\frac{1}{2}$ span, ∴ 5-1 − 2-1 = 1·447 − ·161 = 1·286, and 6-2 = 4 (5-2), ∴ 6-2 = 4 × 1·286 = 5·144 and 7-3 = 6-2. Fig. 76 shows the load lines opened out, but in the reciprocal diagram they will be found to overlie each other, as all the loads and reactions are vertical.

SIZE OF BEAM.

By Anderson's 'Strength of Materials,' p. 209, such a beam would have an ultimate compressive strength of 6000 lbs. per sq. inch, or the safe working stress would equal 600 lbs. per sq. inch in compression, *but no allowance is made there for the transverse stress.*

By the author's 'Handbook for Mechanical Engineers,' p. 54, a fir post up to 10 diameters long may be loaded to $\frac{2}{10}$ tons per sq. inch safely,

$$2240 \times \tfrac{2}{10} = 448 \text{ lbs.}, \qquad \frac{5{\cdot}144 \text{ tons} \times 2240}{448 \text{ lbs.}} = 25{\cdot}72 \text{ sq. ins.}$$

The exact combination of the longitudinal stress and transverse stress would be very complicated, and it would appear to be on the safe side if we assume that the section requires the addition of 25·72 sq. inches to resist longitudinal stress in addition to the 13·1 inches square required to resist the transverse stress.

$13{\cdot}1^2 = 171{\cdot}61$, $171{\cdot}61 + 25{\cdot}72 = 197{\cdot}33$, $\sqrt{197{\cdot}33} = 14{\cdot}05$ ∴ the balk would require to be say . . . . . 14 inches square.

This is a large size for Memel, but, being under 25 feet long, may possibly be obtained. A "clean" piece of Dantzic might be used, care being taken that there were no large knots or shakes near the points of maximum flexure on the tension side, and that the grain was fairly straight. For any greater span or load Fig. 72 should be used. If the timber is required to be wrought all round, Dantzic must be used on account of the size.

TRUSS STRESSES WHEN LOAD IS IN CENTRE OF SPAN.

Draw truss, apportion the loads, and letter it as Fig. 77. Draw reciprocal diagram as Fig. 78, and measure off the stresses, compression in 6-2, 7-3 = 9·644 tons.

9·644 tons at $\frac{2}{10}$ ton per sq. inch = 48·22 sq. inches, while 14 inches square gives over 196 sq. inches, so that the sectional area is ample. There is no transverse stress upon the beam at the centre.

STRESS IN TIE ROD.

By measurement from Fig. 78, 6-5, 7-5, each = 9·94 tons, or by calculation 2·572 − ·161 = 2·411, and $\sqrt{2{\cdot}411^2 + 9{\cdot}644^2}$ = 9·94 tons.

At 5 tons per sq. inch safe load on wrought-iron tie rod, $\frac{9{\cdot}94}{5}$ = 1·99 sq. inches net area = $1\frac{5}{8}$ inch diameter. This would be rather large for a single rod; we may put two rods instead of one, they will then be $\frac{1{\cdot}99}{2}$ = 1 sq. inch net area each, say $1\frac{3}{16}$ inches diameter.

Double rods have the advantage that they may generally be placed altogether outside the timber and therefore obviate the boring of awkward holes such as a single rod necessitates. When they may not project beyond the face of timber a groove may be cut for them, to let them in flush. It is important that they should be tightened up equally that they may divide the strain between them.

The strength of a whole is measured by that of its weakest part, therefore, as the ends are screwed for nuts, the diameter

at the bottom of thread should not be less than the body of the bolt. The area at bottom of thread, Whitworth Standard, is approximately $\frac{d^2 - d}{100}$, where d = diameter of bolt in eighths of an inch. By trial $1\frac{3}{8}$ inch diameter of screwed part is found to give the necessary area, thus $1\frac{3}{8} = \frac{11}{8}$, $\frac{11^2 - 11}{100} = 1{\cdot}1$ sq. inches, as compared with 1 sq. inch area found for rod.

The tie rods will then be $1\frac{3}{16}$ inch diameter with screwed ends $1\frac{3}{8}$ diameter, with double nuts if for rough usage.

The tie rods may either be made in one piece or more. If in one piece there is difficulty with a single rod in threading it through the holes in the timber when more than $\frac{3}{4}$ inch diameter. When in two or more pieces the joints are made by plates or eyes and pins as in Fig. 80.

The tie rods should be parallel so far as possible throughout their length; when drawn closer together in the middle, an increased strain is caused, and tension is put sideways upon the strut.

Screwed ends should be "jumped up" from the rod, to avoid risk of a bad weld in joining on a piece of larger diameter.

STRESS IN STRUT.

By measurement from Fig. 78, and inspection, this equals 4·822 tons, or say 1 sq. inch sectional area of cast iron would be sufficient for resisting direct compression. The strut would, however, be designed for handling and erection, and the dimensions fixed by judgment instead of calculation. The base of strut should be large enough to give sufficient bearing surface against underside of beam without bruising the fibres.

There are very few recorded experiments upon this kind of strain, and none sufficiently definite to be of much practical use. Until these are improved upon, we may take 250 lbs. per sq. inch as being the maximum safe load across the fibres, although in the Chicago Exhibition 600 lbs. was worked to.

$$4{\cdot}822 \text{ tons} \times 2240 = 10{,}801,$$

$$\frac{10{,}801}{250} = 43{\cdot}2 \text{ square inches, area of base of strut.}$$

Assume width 14 inches, then $\frac{43{\cdot}2}{14} = 3{\cdot}1$ inches for the other dimension, or adding $\frac{1}{2}$ inch for a joggle, say 3·6 inches, but there is no reason to stint it,

∴ say . . . . . $14'' \times 4\frac{1}{2}''$

ENDS OF BEAM.

With a single tie rod passing through the beam a flat plate under the nut is sufficient (see Fig. 81), but where the tie rod is double and placed outside the beam, the overhanging plate would bend, and must be replaced by angle iron (see Fig. 82) or special castings (see Figs. 83 and 84).

BEARING SURFACES.

Whether the end be placed in a cast-iron box or not, sufficient bearing surface must be given to the ends of the beam to prevent destruction of the fibres, and, considering how frequently the ends of beams are subject to rot, it will not be wise to restrict the area too much.

Total weight = ·643 + 4·5 = 5·143 tons, half each end $\frac{5 \cdot 143}{2}$ = 2·572 tons, 2·572 × 2240 = 5761·28,

$\frac{5761 \cdot 28}{250}$ = 23 sq. inches, $\frac{23}{14}$ = 1·64 inches wide,

but would not be made less than say 4 inches if covered with cast-iron cap, or a minimum of half the width, say 7 inches for each bearing if left free; and, when resting on timber, generally length = width.

HOLDING DOWN BOLTS.

The beam requires securing in position to prevent displacement. The author's general practice for bolting timber is to let the sectional area of the bolts at each junction in ordinary structures be about $\frac{1}{120}$ of the sectional area of the timber.

$\frac{14^2}{120}$ = 1·63 sq. inches area ∴ say $1\frac{1}{2}$-inch bolt, or two 1-inch bolts placed diagonally, as in Fig. 85, and thereby reducing the probability of splitting the timber or increasing a shake.

PROPORTIONS OF BOLTS FOR CARPENTRY.

Thickness of nut	=	1 diam. of bolt.
„ of head	=	$\frac{3}{4}$ „
Diam. of head or nut over sides .	=	$1\frac{5}{8}$ „
Side of square washer for fir . .	=	$3\frac{1}{2}$ „
Side of square washer for oak . .	=	$2\frac{1}{2}$ „
Thickness of washer . . .	=	$\frac{1}{3}$ „

When the nuts are let in flush in fir, the washers should be same size as for oak.

Head, nut and two washers for fir are equal in weight to

3·6 diam.^{3}, and for oak or flush in fir 2·47d^3. This allows for part of bolt inside nut and washers. Bolt to be measured between the washers, or what is called "wood measure." Weight per inch run = ·22 diam.^{2}

WEIGHT OF STRUCTURE.

	lbs.
Timber $\frac{14^2 \times 23}{144}$ = 31·3 cub. feet at 40 lbs. per foot =	1252
Wrought-iron tie rods, 2 × 1·1 × 3⅛ × 23·5 . =	173
„ 1⅞ nuts, 4 × 5 lbs. each . . =	20
„ angle iron (4″ × 3″ × ⅜″) = 2 × 2·5 sq. inches × 3⅛ × 1′ 7″ long . =	26
Cast-iron strut, at ·26 lbs. per cub. inch . . =	60
Total weight of trussed beam . = (·688 tons)	1539
Wrought iron, Four 1″ bolts 2′ 4″ *w. m.* with nuts and washers	40
„ bridge rail, 48 lbs. per yard × 22 feet .	352
Chocks and spikes, say	20
Total weight of truss and accessories, fixed complete (·88 ton)	1951

COMPLETE DESIGN.

The details having been all arranged, the finished design may now be prepared, as shown upon the diagram. (Figs. 86 and 87.)

PROTECTION FROM DECAY.

If the beam is to be exposed to the weather, creosote is the best protection. The mode of using it, known as Bethell's process, is described in the author's 'Timber Piling in Foundations, &c.' The wood cannot afterwards be painted. All surfaces cut during fitting and erection should be well

tarred, particularly the end grain, and after erection the whole should be tarred. The ironwork should be tarred or painted; the latter is better, but it is sometimes inconvenient when part of the structure is tarred.

COST OF WHOLE TIMBERS.

Timber merchants will supply whole or half timbers in various lengths up to 45 feet at a standard rate (say 1*s.* 9*d.* per cub. foot creosoted) if the *average* length does not exceed 27 feet. Should the average of any parcel exceed 27 feet by any given number of feet, that number will give the number of shillings per load of 50 cub. feet extra charge which will be made. Say average 34 feet, then excess = 7 feet, and price = 7*s.* per load dearer than if average had been 27 feet or under. Approximately, the charge is ¼*d.* per foot on all the timber for each foot the average is in excess of 27 feet.

COST OF ONE BEAM FIXED COMPLETE.

	£	*s.*	*d.*
Creosoted Memel fir, 32 cub. feet at 2*s.* 6*d.* =	4	0	0
Wrought iron in bolts, nuts, washers, tie rods and short pieces angle iron, 2 cwt. 1 qr. 7 lbs. at 20*s.*	2	6	3
Cast-iron strut (exclusive of pattern) 60 lbs. at 2*d.*	0	10	0
Flanged rail and bending, 3 cwt. 0 qrs. 16 lbs. at 10*s.*	1	11	5
Twenty coach screws (½″ × 4″) . . at 3*d.* each	0	5	0
Cleats and spikes	0	2	6
Tarring wood and iron	0	2	6
Hoisting and fixing, say	0	10	0
	£9	7	8

Say £9 10*s.*

BELGIAN STEEL FLANGED RAILS.

When these rails are used in large quantities, the price is considerably less than that given above, e.g. March 1893—

	£	s.	d.	
Steel flanged rails, 40 lbs. per yard and over	3	16	0	per ton.
If with steel fish-plates, add	0	4	0	„
„ iron bolts, add	0	5	6	„
„ iron spikes, add	0	8	0	„

All net per ton f.o.b. Antwerp.

WEIGHT OF ROLLED IRON SECTION.

The weight of any rolled iron section in lbs. per yard run equals the sectional area in sq. inches × 10, and conversely the sectional area in sq. inches = weight lbs. per yard ÷ 10. Mild steel is 2 per cent. heavier than wrought iron.

CHAPTER VII.

CAST-IRON BRIDGE GIRDER.

(See Plate 7.)

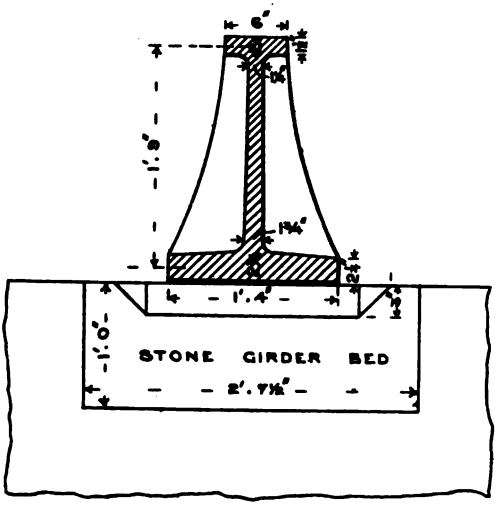
DETAILED calculations and notes with design for:

A cast-iron inside bridge girder (*a*), with flanges of uniform section (*b*), and web of varying depth to give uniform strength (*c*), to carry a uniformly distributed dead load of ½ ton per foot run and a live load of ¾ ton per foot run (*d*), on the bottom flange over a clear span of 20 feet (*e*).

The girder to rest upon "girder beds" or templates of hard sandstone, supported by brickwork in mortar.

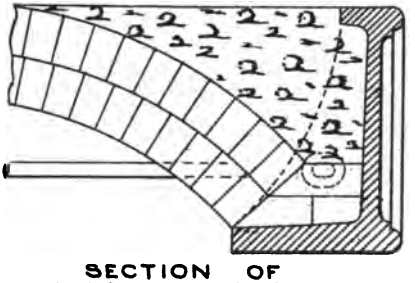
(*a*) Cast-iron girders used for bridges are classed as outside and inside girders. Outside girders are made uniform in depth to carry parapet wall on top flange; have the web near the outer edge of bottom flange to give an equal projection with the top, and have vertical ribs to divide the girder into panels, the edges round the panels being moulded to give effect (see Fig. 91). The bottom flange is secured to the web by angle brackets, webs, or feathers cast on. Inside girders have nothing to carry on top flange, are made of symmetrical cross section, and generally of varying depth (see *c*). An overbridge is a bridge over a railway, and is generally a road bridge, An underbridge is a bridge under a railway; underbridges of cast iron have been prohibited on new lines since August 1883.

PLATE 7



SECTION

Fig. 90



SECTION OF
OUTSIDE GIRDER

Fig. 91

E. & F. N.

Henry Adams

PHOTO LITHO. SPRAGUE & Co LONDON.

(*b*) In patterns for cast ironwork a uniform section is that most easily obtained, and where the section is not constant, it is easier to vary the width than the thickness. In this case, as the depth of web is to be varied the distribution of the stress will require a uniform section in the flanges.

(*c*) As the area of the flanges is made constant, the strength can only be made uniform by varying the depth. When the load is carried by the top flange, it is usual to make both the flange section and the depth uniform, as in girders or bressummers over shop fronts; but for overhead travellers and such like, the parabolic form inverted is frequently adopted, and the girder is called fish-bellied.

(*d*) In ordinary buildings, the whole load upon a girder is considered to be a dead load; but on bridges a large proportion will be a live or moving load, say on overbridges $\frac{1}{3}$ to $\frac{2}{3}$ of the total load. Although the moving loads are applied through the wheels at certain points only, they are considered to be spread uniformly along, except in cases of short girders where the wheel base exceeds half the length of girder, or in transverse girders.

(*e*) Cast-iron girders should not be used for spans exceeding 30 feet.

The minimum allowance for live load on road bridges (overbridges) is 140 lbs. per sq. foot, and a notice must then be put up warning users that the bridge may only be used for loads under 2 tons. The ordinary allowance is 2 cwt. per sq. foot, with a notice warning off loads exceeding 1 ton per wheel, or 5 tons in all. The allowance when heavy traffic is permitted, such as traction engines, road rollers, boiler trollies, &c., is 5 cwt. per sq. foot, but in small span bridges, allowance must be made for concentration of load as wheels

pass centre of span. The usual thickness of metalling is 12 inches.

For further notes on cast-iron girders, see Chapter I.

CALCULATIONS.

Clear span . . . . . 20 feet.

DEPTH OF GIRDER,

assumed to be the effective or mean depth, unless otherwise stated, is the distance between the centres of gravity at middle of the flanges. From $\frac{1}{10}$ to $\frac{1}{15}$ of span, average say $\frac{1}{12}$ = say . . . . . 21 inches.

NET LOAD OR LOAD OVER CLEAR SPAN.

$\frac{1}{2}$ ton per foot run × 20 feet = 10 tons dead load.
$\frac{3}{4}$ „ „ × 20 feet = 15 „ live „

25 tons total.

But for all questions *connected with the stress* in a cast-iron girder, a live load must be considered to distress the metal more than a dead load, in the proportion of not less than 8 to 5. This will also affect the approximate weight of girder, which depends largely upon the stress.

WEIGHT OF CAST-IRON PARABOLIC GIRDER.

W = total distributed dead load in tons;
W′ = „ „ live „ „
l = clear span in feet;
d = mean depth in inches.

Then approximate weight of girder

$$= \frac{(W + 1{\cdot}6\,W')\,l}{290} \times \sqrt{\frac{l}{d}} = \frac{(10 + 1{\cdot}6 \times 15)\,20}{290} \times \sqrt{\frac{20}{20}}$$

$= 2{\cdot}33$ tons.

A fish-bellied girder would be rather heavier than this owing to the longer flange having the larger sectional area. In that case use divisor of 250 instead of 290.

APPROXIMATE SIZE OF FLANGES.

With a ratio of depth to span of $\frac{1}{12}$, the area of bottom flange in sq. inches will approximately be equal to the total distributed dead load in tons, any live load being converted into an equivalent dead load.

Actual dead load . . . . . = 10 tons.
Live load = 15 tons, and
15 × $\frac{8}{8}$ = equivalent dead load . . . = 24 „

Virtual total of dead load 34 „

∴ = 34 sq. inches area, or say, 16″ × 2$\frac{1}{8}$″ or 15″ × 2$\frac{1}{4}$″.

When the load is carried by one side only of the bottom flange, the area of the top is made $\frac{1}{3}$ the area of the bottom but when both sides are loaded equally, or load rests on top flange, then top flange is usually made $\frac{1}{4}$ the area of bottom.

SAFE LOAD ON BEARING SURFACE.

(See p. 43, Chapter IV.)

Hard sandstone, say York . .	12 tons	per foot super.
Stock brickwork in mortar . .	4 „	„

CHAMFER ON STONE BEDS.

Say 1$\frac{1}{2}$ inches per 10 feet span = . . . 3 inches.

LOAD TO EDGE OF BEARING SURFACES

= load tons per foot run × (clear span + chamfers) added to weight of girder,

$$(\cdot 5 + \cdot 75)(20 + \cdot 25 + \cdot 25) + 2 \cdot 33 = 27 \cdot 955$$

say . . . . . 28 tons.

WIDTH OF BEARING SURFACE.

This will be not less than width of bottom flange but may be more if necessary, as the bottom flange may be widened out near the ends to any desired width,

say . . . . . 15 inches.

LENGTH OF BEARING SURFACE.

(See p. 5, Chapter I., and p. 44, Chapter IV.

$$l = \frac{W}{s\,b - w} = \frac{28}{(12 \times 1\tfrac{1}{4}) - 1 \cdot 25} = 2 \cdot 04,$$

$\frac{2 \cdot 04}{2} = 1 \cdot 02$, say . . . . 1 foot each end.

TOTAL LENGTH OF GIRDER.

$20 + \cdot 25 + \cdot 25 + 1 + 1 =$ . . . . 22·5 feet.

EFFECTIVE SPAN

equals the distance from centre to centre of the bearing surfaces. Clear span, 20 feet + 2 chamfers ·5 feet + one bearing surface 1 foot = ... 21·5 feet.

EFFECTIVE LOAD AND MAXIMUM STRESS.

Where a cast-iron girder is subject to a live load, the flanges must be calculated by a special method.

The usual formula for stress in bottom flange under distributed load, $\frac{w\,l^2}{8\,d}$ or $\frac{\mathrm{W}l}{8\,d}$ is only applicable to the case of a dead load, such as would come upon the girder in an ordinary building, allowing a safe working stress of 1½ tons per sq. inch. In the present case, a formula embodying the action of a live load must be taken.

AREA OF BOTTOM FLANGE.

The author's rule for area of bottom flange is as follows :— Effective span in feet × (5 times distributed dead load in tons + 8 times distributed live load in tons), divided by 5 times the mean depth in inches :—

Effective span = 21·5 feet.
Distributed dead load = (·5 × 21·5) + 2·33 = 13·08.
„ live load = ·75 × 21·5 = 16·25.

$$\frac{21{\cdot}5 \times \{(5 \times 13{\cdot}08) + (8 \times 16{\cdot}125)\}}{5 \times 21} = 39{\cdot}8, \text{ say } 40 \text{ sq. in.}$$

By the Board of Trade Regulations the breaking weight must not be less than 3 times the dead load + 6 times the live load. This will give :—

$$\frac{\left\{\begin{array}{l}3 \text{ times dead load tons distributed} \\ +\,6 \text{ times live } \quad „ \quad „\end{array}\right\} \times \text{effective span feet}}{8 \text{ times mean depth ft.} \times \text{ult. tensile strength tons per sq. in.}}$$

= sq. inches sectional area in bottom flange.

$$\frac{\{(13{\cdot}08 \times 3) + (16{\cdot}25 \times 6)\} \times 21{\cdot}5}{8 \times 1{\cdot}75 \times 7{\cdot}5} = 28 \text{ say } 30 \text{ sq. in.}$$

while we have provided 40 sq. inches, or 33 per cent. more than the law requires, but not more than prudence dictates, seeing that these girders are so placed as to prevent periodical inspection and cleaning.

A common rule in some offices, founded upon the Board of Trade Regulations, is the following :—

$$\frac{\frac{1}{4}\text{ span (3 times dead load + 6 times live load)}}{\text{depth inches}} = \text{s. a. bottom flange.}$$

This assumes 6 tons per sq. inch instead of $7\frac{1}{2}$, as the maximum strength, and gives 35 sq. inches for area of flange, being about midway between the other results.

It should, however, be noted, that some cast-iron girders erected under the sanction of the Board of Trade, have failed, and many others have been removed to prevent failure, therefore keep to the larger area.

ACTUAL SIZE OF FLANGES.

BOTTOM FLANGE should have a width of $\frac{3}{5}$ to $\frac{4}{5}$ the depth, and a thickness of $\frac{1}{6}$ to $\frac{1}{8}$ the width, but never less than $\frac{1}{12}$ width.

Depth 21 inches, $\frac{3}{5}$ of 21 = 12·6, $\frac{4}{5}$ of 21 = 16·8,

∴ width say . . . . . 16 inches.

Width 16 inches, $\frac{1}{12}$ of 16 = 1·33, $\frac{1}{8}$ of 16 = 2, $\frac{1}{6}$ of 16 = 2·66, but thickness must be $= \frac{\text{area}}{\text{width}} = \frac{40}{16} = 2{\cdot}5$,

∴ mean thickness say . . . . . $2\frac{1}{2}$ inches.

TOP FLANGE should have an area = $\frac{1}{4}$ bottom flange (see Chapter I., p. 10), and a thickness = $\frac{1}{3}$ to $\frac{1}{4}$ its width.

$$\frac{40}{4} = 10 \text{ sq. inches area.}$$

Make it, say . . . . . 6 inches wide;

then $\frac{6}{3} = 2''$, or $\frac{6}{4} = 1\frac{1}{2}$ for thickness, but $\frac{10}{6} = 1{\cdot}66$,

say . . . . . $1\frac{3}{4}$ inches

mean thickness, which will be a very good proportion.

When the load is carried by one side only of the bottom flange, the area of the top is made $\frac{1}{3}$ the area of the bottom.

WEB.

In a cast-iron girder the thickness of web is made proportional to the thickness of flanges, and not to the shearing stress. The web should be $\frac{2}{3}$ to $\frac{3}{4}$ the thickness of the flanges and tapered throughout its depth towards the top. Curved fillets should be put in all angles of a radius of $\frac{1}{2}$ to $\frac{2}{3}$ thickness of parts joined.

Top flange $1\frac{3}{4}$ inch thick, $\frac{2}{3}$ of $1\frac{3}{4} = 1{\cdot}17$, $\frac{3}{4}$ of $1\frac{3}{4} = 1{\cdot}31$,

Bottom flange $2\frac{1}{2}$ thick, $\frac{2}{3}$ of $2\frac{1}{2} = 1{\cdot}7$, $\frac{3}{4}$ of $2\frac{1}{2} = 1{\cdot}9$,

$\therefore$ make web say . $1\frac{1}{4}$ thick at top and $1\frac{3}{4}$ thick at bottom.

For fillets, $\frac{1}{2}$ of $2\frac{1}{2} = 1{\cdot}25$, $\frac{2}{3}$ of $1\frac{3}{4} = 1{\cdot}17$, say $1\frac{1}{4}$ inch radius at bottom; and $\frac{1}{2}$ of $1\frac{3}{4} = {\cdot}875$, $\frac{2}{3}$ of $1\frac{1}{4} = {\cdot}833$, say $\frac{7}{8}$ inch radius at top; or all the fillets may be 1 inch radius for sake of uniformity.

GENERAL RULE IN DESIGNING CAST-IRON GIRDERS.

In calculating a cast-iron girder, if any part works out too thin the proportions must be altered; no part should be less in thickness than $\frac{1}{12}$ of its width, or than $\frac{1}{2}$ to $\frac{1}{3}$ the thickest part of the girder.

ELEVATION OF GIRDER.

The girder is to be of uniform strength, obtained by reducing the depth towards the ends. The theoretical curve

will be a parabola passing through centre of gravity of top flange, with a base along the centre of gravity of the bottom flange, the effective span giving the length of base; but sufficient sectional area must be allowed over the points of support to resist the shearing stress.

$$\frac{13{\cdot}08 + 16{\cdot}25}{2} = 14{\cdot}66$$

tons shearing stress at top of chamfer on abutment. $\frac{14{\cdot}66}{1{\cdot}5} = 9{\cdot}77$ sq. inches, but the flange alone, in which there is no stress at this point, will greatly exceed this area, and therefore the only consideration is a practical one of foundry work, make girder say 7 inches deep at this point.

The curve of top flange may be drawn to individual taste so long as the parabola is included within the area. The ends are sometimes made $\frac{2}{3}$ of the central depth.

DEFLECTION IN CENTRE UNDER FULL LOAD.

W = effective load tons distributed;
s = effective span in feet;
d = mean depth in inches.

$$\frac{{\cdot}0005\,W s^2}{d} = \frac{{\cdot}0005 \times 29{\cdot}33 \times 21{\cdot}5^2}{21} = {\cdot}322,$$

say . . . . . $\frac{3}{8}$ inch.

By Molesworth (p. 169, 21st edition) safe deflection $= \frac{1}{40}$ inch per foot span under test load of $\frac{1}{3}$ breaking weight, but this is considered to be rather a full allowance.

CAMBER.

Usual camber for cast-iron girder, $\frac{3}{4}$ inch per 10 feet clear span.

$\frac{20 \times \cdot 75}{10} =$ . . . . . $1\frac{1}{2}$ inches.

STIFFENERS OR FEATHERS.

These should be an even number, about equally divided over the girder 4 to 5 feet apart, same thickness as upper part of web, and carried out to extreme edge of each flange. Sometimes small intermediate feathers, 3 or 4 inches deep, are put in.

AREA OF STONE FOR BEDDING GIRDER.

16″ × 12″ = 192 sq. inches, each bearing surface of girder,

$\frac{192 \times 12 \text{ tons on stone}}{4 \text{ tons on brick}} = 576$ sq. inches area of each stone.

WIDTH OF STONE.

3 inch chamfer + 12 inch girder + say 3 inch margin beyond girder = ... 1 foot 6 inches.

LENGTH OF STONE.

1 foot 6 inches = 18 inches, $\frac{576}{18} = 32$ inches; but should be a multiple of $4\frac{1}{2}$ inches or 9 inches to suit brickwork. Make it, say $31\frac{1}{2}$ inches = . . . 2 feet $7\frac{1}{2}$ inches.

THICKNESS OF STONE (see p. 56).

$\frac{6 \times 20}{15} + 4 =$ . . . . . 12 inches thick.

SIZE OF STONE.

$2' 7\frac{1}{2}'' \times 1' 6'' \times 1' 0''$.

Girder bedded on two thicknesses of tarred or asphalted roofing felt, with sheet of 8 lb. lead between them.

ESTIMATE OF WEIGHT.

		Lbs.
Top flange, parallel part .	$21' 2'' \times 6'' \times 1\frac{3}{4}''$	= 814
Add for ends . .	$2' 0'' \times 11'' \times 1\frac{3}{4}''$	
Bottom flange . .	$22' 6'' \times 1' 4'' \times 2\frac{1}{2}''$	= 2812
Web	$22' 3'' \times 1' 0'' \times 1\frac{1}{2}''$	= 1251
Stiffeners . . .	$4 \times 1' 0'' \times 9'' \times 1\frac{1}{4}''$	= 140
Fillets	116 feet run	= 116
		5133

At $37\frac{1}{2}$ lbs. per foot super 1 inch thick, the weight = 5133 lbs. = 2·291 tons, and the approximate weight having been taken at 2·33 tons the designing may be proceeded with.

ESTIMATE OF COST, INCLUDING ORDINARY PROFIT.

	£	s.	d.
Iron and labour at £7 10*s.* per ton .	17	3	8
Erection at 15*s.* „ .	1	14	4
Total cost fixed .	18	18	0

Exclusive of any brickwork or masonry, of cost of pattern, or of any charge for testing. The actual cost will of course vary with the market value of iron and labour, and with the number of girders. Approximate cost of cast-iron girder $£ = \cdot 038\ W\,l.$

TESTING CAST-IRON GIRDERS.

Usually tested in pairs to $\frac{1}{3}$ of breaking weight, with top flanges facing each other and wrought-iron straps passed round the ends at distance = clear span, a hydraulic press with gauge attached being placed in centre between them. The whole being horizontal the weight of girders does not affect the result. The deflection is taken by straining a string or fine wire from end to end of each girder and measuring the deflection in the centre, when the gauge indicates any given load. It should not exceed $\frac{\cdot 001 \times \text{span ft.}^2 \times \text{load tons centre}}{\text{depth inches}}$, although it is generally very irregular. The actual deflection $\times \frac{5}{8}$ will equal the deflection which would be produced by a similar load spread equally over the girder. The permanent set should not exceed $\frac{1}{3}$ the maximum deflection with the full test load.

CHAPTER VIII.

(See Plate 8.)

CAST-IRON STANCHION OR STORY POST, 10 FEET HIGH, TO CARRY A DEAD LOAD OF 36 TONS.

FORM OF SECTION.

IN order to judge whether a + or H section will be more suitable, we must ascertain roughly what area will be required.

The H section will give an increased area over the + section as 3 to 2 for the same over-all dimensions.

When the stanchion is required against a wall, a ⊔ or ⊔⊔ section is often adopted, flush with face of wall, and a bracket on top projecting from the wall to carry girder. In these cases the centre line of load seldom coincides with the axis through centre of gravity of section of stanchion, and there is great probability of the ribs being put in tension, if the plain side is put outwards.

DIAMETER OF WIDTH OF SECTION

should be from $\frac{1}{12}$ to $\frac{1}{20}$ of the length.

THICKNESS OF METAL

should be from $\frac{1}{6}$ to $\frac{1}{8}$ of the diameter.

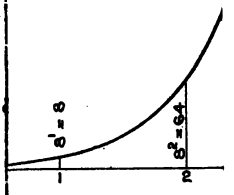
FRACTIONAL POW

NUMBER WITHOUT

Fig. 9 LOGARITHMS

Fig. 92

Fig.



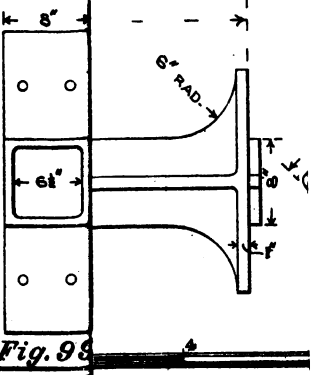


Fig. 9

APPROXIMATE SAFE LOAD ON COLUMNS AND STANCHIONS.

Up to 8 diameters long = 5 tons per sq. inch.
„ 10 „ = 4 „ „
„ 12 „ = 3 „ „
„ 15 „ = $2\frac{1}{2}$ „ „
„ 18 „ = 2 „ „
„ 24 „ = $1\frac{1}{2}$ „ „
„ 30 „ = 1 „ „

APPROXIMATE AREA.

Say 15 diameters long, 36 tons at $2\frac{1}{2}$ tons per sq. inch $= \frac{36}{2\frac{1}{2}} = 14{\cdot}4$ sq. inches, or say + section 8″ × 8″ × 1″ thick $(8 + 8 - 1) \times 1 =$ say . . . 15 sq. inches area.

SUFFICIENCY OF AREA.

We will now test by independent methods whether 15 sq. inches will give sufficient strength for our stanchion.

FACTOR OF SAFETY

should be $\frac{1}{6}$ for short pillars and $\frac{1}{10}$ for long pillars, those being called long which exceed 25 diameters in length. Box considers that with an invariable dead load, a factor of safety of $\frac{1}{3}$ allows sufficient margin.

STRENGTH BY GORDON'S FORMULA.

W = crushing weight in tons;
s = sectional area in sq. inches;
d = diameter over arms in inches;
l = length in inches.

$$W = \frac{36s}{1 + \frac{3l^2}{400d^2}}$$

$$W = \frac{36 \times 15}{1 + \frac{3 \times 120^2}{400 \times 8^2}} = 201 \text{ tons crushing weight.}$$

Safe load = 201 ÷ 6 = ... 33·5 tons.

STRENGTH BY TIMMINS' TABLES.

8″ × 8″ × 1″ = 15 sq. inches area = 46·8 lbs. per foot run, and for 10 feet length, safe load = . . 20·1 tons.

STRENGTH BY BOX'S TABLES.

Breaking weight in tons.

8″ × 8″ × 1″ × 10 feet long = 261 tons.

$\frac{261}{6}$ = safe load of say . . . . . 43½ tons.

STRENGTH BY TABLE IN LOCKWOOD'S ENGINEERS' POCKET BOOK.

Safe load in cwts.

8″ × 8″ × 1″ × 10 feet long = 354 cwt.

or $\frac{354}{20}$ = ... 17·7 tons.

STRENGTH BY HODGKINSON'S FORMULA.

(Box, 'Strength of Materials,' p. 120.)

$$\frac{M_p \times (b^{2\cdot6} \times t) + \{t^{2\cdot6} \times (b - t)\} \text{ inches}}{L^{1\cdot7} \text{ feet}} \cdot$$

= B W tons by flexure.

Which must not exceed $\frac{1}{4}$ of the sectional area × ultimate crushing strength (cast iron = 42 tons) per sq. inch in order to provide against incipient crushing.

$$8^{2\cdot6} * = 223, \qquad 1^{2\cdot6} = 1, \qquad 10^{1\cdot7} = 50\cdot1.$$

M_p = constant multiplier for cast iron.

75 both ends flat.
50 one end flat.
25 both ends pointed.

$$75 \times \frac{(223 \times 1) + (1 \times 7)}{50\cdot1} = 344 \text{ tons B W}$$

but $$\frac{15 \text{ sq. inches} \times 42 \text{ tons}}{4} = 157\cdot5 \text{ tons,}$$

which is the maximum efficient strength, and safe load =

$$\frac{157\cdot5}{6} = \quad . \quad . \quad . \quad . \quad . \quad . \quad 26\cdot25 \text{ tons.}$$

* TO FIND THE FRACTIONAL POWER OF A NUMBER WITHOUT LOGARITHMS.

Upon a base line (see Plate 8, Fig. 92) set off equal divisions, say inches; at end of first division set up an ordinate to scale equal to the number of which the power is required, e. g. 8. At the end of the second division set up the square of the number, at the end of the third set up the cube, and draw a curve through the points found. Then for 1·7 power mark off 1·7 inches on the base line and scale off the vertical height to curve, or for 2·6 power set off 2·6 inches and scale off the height. This method is convenient for small whole numbers, but for others it is better to make the slight effort required to master and use a table of logarithms.

VARIATION OF THICKNESS.

Other things being the same, the strength of a stanchion varies directly as the thickness of the metal.

RELATIVE STRENGTH.

A stanchion of + section is much less economical than a hollow cylindrical column, so far as the use of the metal goes. For instance a cylindrical column of the same external diameter as the stanchion and containing the same sectional area, will have very nearly double the strength. There are, however, advantages in certain cases in employing stanchions.

COMPARISON OF STRENGTH BY THE VARIOUS RULES

	Tons.
By approximate rule	37·5
„ Gordon's formula	33·5
„ Timmins' Tables	20·1
„ Box's Tables	43·5
„ Lockwood's Tables	17·7
„ Hodgkinson's formula . . .	26·25
Average of above	29·76

This is a very unsatisfactory result, and we must refer to experiment for confirmation of our formulæ.

HODGKINSON'S EXPERIMENTS.

Hodgkinson found that a cast-iron pillar of + section 3″ × 3″ × ·48″, and 7·562 feet long, both ends rounded, broke with 17,578 lbs. = 7·8475 tons.

COMPARISON OF EXPERIMENTAL STANCHION WITH PREVIOUS RULES.

Allowance of 3 to 1 being made for difference between flat or fixed and rounded or jointed ends.

		Tons.
By actual loading . . .	=	7·8475
„ approximate rule assuming $\frac{1}{6}$ factor of safety.	=	5·300
„ Gordon's formula . . .	=	3·353
„ Timmins' Tables, say . .	=	4·063
„ Box's Tables		—
„ Lockwood's Tables		—
„ Hodgkinson's formula . .	=	7·0125

SUMMARY.

It will thus be seen that there is considerable variation, comparing the rules with actual experiment; but on the whole we shall feel tolerably safe in assuming that our section of 8″ × 8″ × 1″ = 15 sq. inches will be sufficient for carrying the load of 36 tons with safety. The need for further experiments on a large scale, however, is very evident.

FEATHERS IN ANGLES.

It is customary to cast intermediate flanges, feathers, webs, or stiffeners (see A, Figs. 93 and 94, Plate 8) in the angles at intervals of 3 to 6 feet. These should be from the same thickness of metal as the general section to $\frac{3}{4}$ of this thickness, and rounded on the edge.

JOINT IN LENGTH.

When from any cause the length is considered too great for one casting, a joint may be made half-way up, but it must be accurately fitted and bolted, say, ends planed and bolted as shown in Figs. 95 and 96.

FILLETS IN ANGLES.

All castings require curved fillets in the angles, to avoid the weak place formed by the incomplete junctions of the crystals, which occurs in the cooling of sections connected by a square angle. A common but bad form is shown in Figs. 97 and 98. The fillets should be of a radius = $\frac{1}{2}$ to $\frac{3}{5}$ of parts joined.

Say 1-inch metal, then fillets = ... $\frac{1}{2}$" radius.

ENDS OF STANCHION.

The ends must be sufficiently spread to enable them to be securely fixed, they must be well bedded, and guarded from the effects of unequal settlement or subsidence of foundations.

The area will depend upon the material they support or upon which they rest.

Assume stanchion standing on York stone, the top supporting a floor by two 12" × 9" beams and carrying above it another stanchion bearing half the load.

BOTTOM FLANGE.

Load 36 tons + weight of stanchion (say ½ ton) = 36½ tons.

York stone—crushing commences at 250 tons per sq. foot safe dead load $\frac{1}{10}$ = 25 tons per sq. foot, as the greatest dead load in an extreme case. (See p. 43 for area of railway girder beds.)

$\frac{36{\cdot}5}{25}$ = 1·46 sq. ft., 1·46 = 1·21 × 1·21 (or $\sqrt{1{\cdot}46}$ = 1·21)

or say, bottom flange . . . . 1′ 3″ × 1′ 3″

The ribs of the stanchion should be curved outwards to meet the corners of the base, and a hole for a 1-inch bolt put between each, or cleats cast on bottom as shown in Fig. 100.

TOP FLANGE.

Say 8″ × 8″ square box or stilting ¾ inch thick, and 12 inches deep, on top of stanchion, with or without flange, to receive foot of upper stanchion, and bracket each side to carry floor beams.

Load on upper stanchion 18 tons, 8 × 8 = 64, 6½ × 6½ = 42·5, 64 − 42·5 = 21·5, $\frac{18 \text{ tons}}{21{\cdot}5 \text{ sq. inches}}$ = ·837 tons per sq. inch on box, giving ample margin, but it is not desirable to reduce it, as the different parts of a casting should be so proportioned as to cool at a uniform rate.

Load from floor beams $\frac{18}{2}$ = 9 tons each, width of bracket 8 inches, then at 250 lbs. per sq. inch safe load (see p. 91 *ante*)

$\frac{9 \text{ tons} \times 2240}{250 \text{ lbs.} \times 8 \text{ inches}} = 10{\cdot}08$, say 10 inches long, or each projecting bracket 8″ × 10″ with two holes for ¾-inch bolts in each.

DESIGN.

The working drawing may now be proceeded with as shown in Figs. 99 to 102.

Stanchions of this pattern have rather a bare look, and improvement in the appearance may be made by forming the upper end as shown in Fig. 103, but it would have no advantage in strength.

COST OF STANCHION.

This will depend very largely upon the number required, and whether made in London or the country. In the following sections an attempt has been made to analyse the cost.

COST OF WOOD FOR PATTERN.

Pattern-makers' pine is the American yellow pine (*Pinus strobus*); it is a pale straw colour when freshly cut, but turns brown when exposed to the air; it has very few knots, is clean and straight in the grain, soft and uniform, and marked with short dark lines, is light in weight and obtained of a very large size, logs say 2 feet square; takes glue well (and also putty in the hands of apprentices). It is very subject to dry rot.

Cost, £60 per 120 of 12 feet × 3″ × 11″ = 10*s.* per plank, or say 4*s.* per cubic foot sawn to dimensions.

Although this is the general wood for patterns, stanchions are frequently made of Baltic yellow deal (*Pinus sylvestris*), the cost of which per foot super. of 1 inch stuff would be found as follows:—

	£	s.	d.
3 inch Yw. Xna. deals at per standard (12 feet × 3″ × 9″)	34	10	0
Add cartage, &c.	1	10	0
÷ Standard or long hundred = 120)	36	0	0
∴ each =		6	0
Add sawing to 1 inch (2 cuts) . . .			7½
		6	7½
Add profit 15 per cent.		1	0
÷ No. of boards out of one deal = . .	3)	7	7½
÷ average length feet each piece = .	12)	2	6½
Cost per foot lineal, 9 inches wide . .			2½
Add ⅓ for difference between 9 inches and 12 inches = ...			1
Cost per foot super, 1 inch thick . .			3½

In the same way 1½-inch stuff will be found worth 5*d.* per foot super, 1¼-inch worth 4¼*d.*, and ⅝-inch worth 2⅜*d.*

These are nominal thicknesses only, and are always actually less owing to the width of the saw kerf. When planed, the thickness may vary from ⅛ inch to ¼ inch less than the nominal, and the stuff must therefore be priced at ¼ inch thicker than the finished sizes.

Arris strips for fillets at ½*d.* per foot run.

COST OF PATTERN.

A pattern-maker's rate of pay is 9*d.* per hour, but the actual pay varies from 5*s.* upwards per day according to district and circumstances.

Such work as making a stanchion pattern would occupy on the average about half an hour per foot super of the stuff used, with $\frac{1}{12}$ hour additional per foot run of all rounded or shaped edges and filleted angles.

The box on the top of stanchion could not be moulded hollow, and therefore it would be closed in and a "print" put on the end to make an impression in the sand to support the end of a "core," the weight of the other part being borne by a "chaplet." A very simple "core box" like a brick mould would suffice, into which the sand could be rammed and the edges of the core trimmed off after it was dry.

			Material.		Labour.	
			s.	*d.*	*s.*	*d.*
3′ 3″	sup. $\frac{3}{4}$″ thick (1″ nominal) at	3½*d.*	1	1	—	
20′ 2″	„ 1″ „ (1¼″ „) „	4¼*d.*	7	1½	—	
23′ 5″	„ for screws and nails „	¼*d.*	0	6	—	
23′ 5″	sup., ÷ 2 = 11½ hours „	9*d.*	—.		8	7½
56′ 10″	run arris fillets . „	½*d.*	2	4½	—	
39′ 5″	rounded and shaped edges		—		—	
96′ 3″	× $\frac{1}{12}$ = 8 hours . „	9*d.*	—		6	0
	Material .		11	1½	—	
	Labour .		—		14	7½

Total net cost of pattern, say £1 5*s.* 9*d.*

WEIGHT OF STANCHION

may be taken approximately equal to 15 times the weight of the pattern, but in all castings this depends very much upon the amount of "coring."

Taking out the weight in detail from the dimensions on the drawings, and allowing for the curved angles, it will be found (at 0·263 lbs. per cubic inch of metal) to be . . 0·31 ton.

RELATIVE COST OF CASTINGS.

	Per cwt. s.	d.
Split pulley castings	12	0
Ordinary do. and loam castings . .	10	0
Engine, pump, forge and general shop machine castings	9	0
Crane, press, mill and steam hammer castings	8	6
Stone-crushing and brick-making machines	8	0
Mortar mill castings	7	6
Wagon wheel, pipe, column, girder and stanchion castings	6	6
Tank plates and firebars . . .	6	0

ESTIMATE OF THE COST OF CASTING.

Description of work	+ stanchions. Per ton. £ s. d.
Material :—	
Metal	3 0 0
Waste	0 5 0
Cupola coke	0 2 0
Stove coke	0 0 0
General stores	0 5 0
Labour :—	
Moulding	1 5 0
Cleaning and trimming . . .	0 5 0
General labourage	0 7 6
Total net cost .	5 9 6
General expenses	0 17 6
Bad castings and risk	0 2 6
Total cost per ton at the foundry .	6 9 6

Say £6 10*s.* per ton.

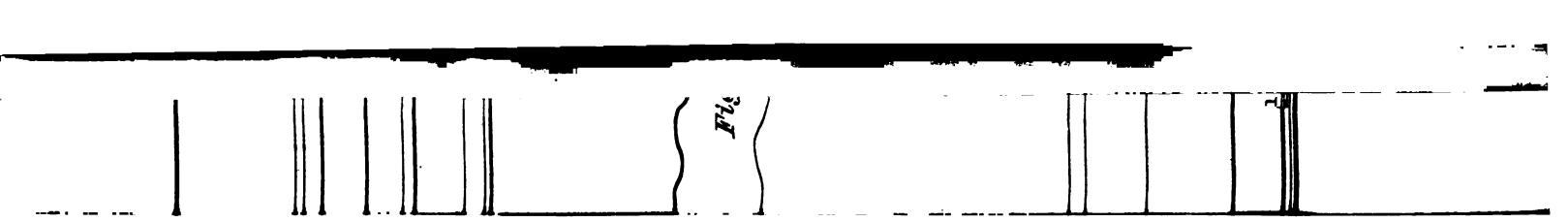
COST OF STANCHION.

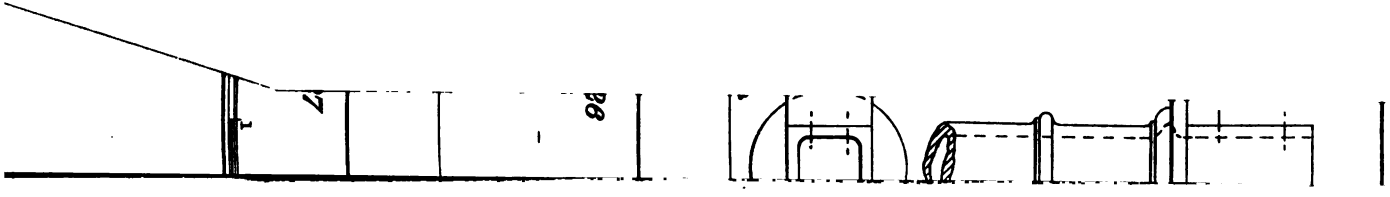
Exclusive of cost of pattern.

	£ s. d.
0·31 tons at £6 10*s.*	2 0 3
Machine work and fitting, say . . .	0 7 6
Add for delivery at 10*s.*	0 3 1
„ erection at 10*s.*	0 3 1
„ painting 3 oils, 4 yards at 10*d.* .	0 3 4
	2 17 3

Say £2 17*s.* 6*d.* each.

ns.





CHAPTER IX.

CAST-IRON COLUMN.

(See Plates 9 and 10.)

CAST-IRON HOLLOW COLUMN 15 FEET HIGH, TO CARRY A DEAD LOAD OF 30 TONS.

COLUMNS may have the ends rounded or flat, with or without base plates or bolts.

Columns with rounded ends were used in supporting the roof of the National Agricultural Hall (Olympia), see Fig. 104, in order to prevent a bending moment from the unequal loading of the gallery floors.

Columns with flat ends and detached base plates were used on the London and North Western Railway at Watford Viaduct, see Figs. 105 and 106, to lessen the risk of breakage from initial strains in the casting.

A more usual mode of forming detached base plate in heavy work is shown in Figs. 107 and 108; similar columns were used on the Great Eastern Railway under some of the suburban bridges.

The cap and base of large columns are frequently cast in separate pieces; being for ornament only they can thus be made much lighter. The columns under the North London Railway bridge at Randolph Street, Camden Town, were made in this way. See Figs. 109 to 115.

Columns with base plates cast on, and with or without

bolts, are generally used for all cases. When the length does not exceed ten times the diameter the bolts are frequently omitted.

When the columns support bridge girders, the tops are usually flat and bolted to the girders; but in buildings when iron or timber beams are supported, they either rest on brackets or in pockets, or pass through a box cast on top of the column, so that the columns vertically over each other may be in direct contact from top to bottom of building.

MODE OF STRAINING.

"Approximately, so long as the length of a compression bar, pillar, or strut of cast iron does not exceed five times its least dimension, it tends to fail by direct crushing; while if its length exceeds this proportion, it tends to fail by bending sideways and cross breaking. In wrought iron the proportion of length to least dimension, up to which failure tends to take place by direct crushing, is approximately 10, while in dry timber it is approximately 20. It is possible that steel of average hardness would occupy an intermediate place between cast and wrought iron, but there is at present no definite information on the subject."—*Wray's Theory of Construction.*

For circular columns fixed at both ends Reuleaux makes the limit 20 for cast iron, and 48 for wrought iron.

Gordon's investigations showed that when the length is 26·4 times the diameter, pillars, columns, or vertical struts are of equal strength, whether of wrought or cast iron; when shorter, cast iron is stronger, when longer, wrought iron is stronger.

From Kirkaldy's experiments it appears that cast iron pillars under 5 diameters long failed entirely by crushing, from

5 to 20 diameters partly by crushing, partly by bending, over 20 diameters entirely by bending.

Rondelet's experiments showed that cast-iron pillars do not yield by flexure when the length is less than 10 times the least lateral dimension.

CLASSIFICATION OF PILLARS (*Moos*).

Class.	Name.	Material Crystalline.	Fibrous.
I.	Very short .	$l/d = < 1{\cdot}5$	$< 1{\cdot}5$
II.	Short .	„ 1·5 to 5	1·5 to 10
III.	Long .	„ 5 to 15	10 to 30
IV.	Very long .	„ over 15	over 30

When the length approaches 30 times the diameter, the column will probably form part of a braced framework, in which case the bracing members, by attachment to some intermediate point, will assist the stability; but, unless the bracing be of such a character as to absolutely fix the column at such intermediate point and prevent bending *in either direction*, the full length must be considered in calculating the strength.

DIAMETER OF COLUMN.

The diameter will seldom exceed $\frac{1}{10}$ of the length, and should never be less than $\frac{1}{30}$. The proportion should vary with the character of the structure and be in keeping with it. As a general rule the diameter will depend to a great extent upon the load to be carried; if made large in proportion the thickness of metal may be insufficient to allow for contingencies in casting, and if small the ratio of length to diameter

will render the column unnecessarily liable to fail by bending. It should be so proportioned as to involve a reasonable thickness.

THICKNESS OF METAL

may vary from $\frac{1}{8}$ to $\frac{1}{16}$ of the outside diameter, but should in general be about $\frac{1}{12}$.

By another rule it is made to vary with the length, the minimum thickness in inches being $\frac{\text{length in feet}}{24} + \cdot 25$, and the maximum thickness $1\frac{1}{2}$ inches.

Columns of 6 inches diameter or over, should as a general rule not be less than $\frac{3}{4}$ inch thick, and under 6 inches diameter not less than $\frac{1}{2}$ inch thick.

UNEQUAL THICKNESS OF CASTING

is generally caused by the displacement of the "core," so that the metal is thinner on one side than the other. Hodgkinson* says with reference to this, "It is gratifying to find that a matter which would seem to destroy all confidence in a pillar does not produce a great reduction in the strength;" his experiments leading him to suppose that the thinner side always takes up the compressive strain, and the thicker the tensile, when the loading is exactly central.

SECTIONAL AREA.

The chief calculation is to ascertain the necessary sectional area of the shaft of the column. This depends partly upon the ratio of diameter to length, and partly upon the ratio of thickness to diameter.

* Phil. Trans., 1840.

With given dimensions the sectional area may be obtained either by deducting the area due to the inside diameter from that due to the outside diameter taken from a table of areas, or the mean diameter (i. e. outside diameter minus one thickness) may be multiplied by the thickness and by $\pi = 3{\cdot}1416$.

When $t = \frac{1}{12} d$, then sectional area in square inches =

$$\frac{\pi}{4}\left\{d^2 - (\tfrac{5}{6}\,d)^2\right\} = \frac{\pi}{4}(\tfrac{11}{36}\,d^2) = {\cdot}24\,d^2.$$

ULTIMATE RESISTANCE OF CAST IRON TO CRUSHING.

Rankine* says average = 112,000 lbs. = 50 tons, but this is too high for practical use in any case.

With metal of good average quality we may fix it at 42 tons = 94,080 lbs., but to ensure the requisite quality a sample bar should be tested.

In the absence of tested sample bars 36 tons is the greatest resistance which should be assumed.

TEST OF SAMPLE BARS.

Three bars to be cast in dry mould from each melting, 3 feet 6 inches long, 2 inches deep, 1 inch wide, 3 feet between bearings, to break with an average of 30 cwt. in centre, or minimum of 28 cwt., and to deflect not less than $\frac{8}{10}$ inch, with load of 25 cwt. applied direct without intervention of levers.†

* 'Civil Engineering,' p. 769.

† See the author's 'Handbook for Mechanical Engineers,' p. 64.

FACTOR OF SAFETY.

If the calculation be made for breaking weight a factor of safety of not less than 6 should be taken for a dead load, and 8 for a live load.

If loading be uneven, as for a column in centre of erecting-shop supporting a travelling crane on either side, a factor of safety of 10 should be allowed.

If the column will be in such a position as to be liable to collision with vans, extra strength must be allowed or fender stones put against base.

J. B. Francis, of Lowell, Mass., in his 'Tables of Cast-Iron Pillars,' recommends that in order to allow for unequal loading, imperfect casting, bad end bearings, side blows, &c., the safe load should not be taken at more than $\frac{1}{15}$ of Hodgkinson's breaking load, if the pillars are roughly cast and the ends not perfectly planed and adjusted; and $\frac{1}{5}$ when they are so, and the loads about equally distributed.

Prof. Unwin says that when the load on a column is $\frac{1}{16}$ of diameter out of the centre, the bending stress on column is increased 50 per cent.

Stoney says long pillars irregularly fixed lose from $\frac{2}{3}$ to $\frac{4}{5}$ of their strength.

AUTHOR'S RULE FOR LOADING COLUMNS.

For a thickness of $\frac{1}{12}$ diameter:—

10 to 15 diameters long = 4 tons per sq. inch.
15 „ 20 „ „ = 3 „ „
20 „ 25 „ „ = 2 „ „
25 „ 30 „ „ = 1½ „ „

('Handbook for Mechanical Engineers,' p. 54.)

This is founded upon the following formula, where r = ratio of length to diameter.

Safe load tons per sq. inch $= \frac{12\,t}{d}\left\{5{\cdot}8 - \frac{r}{5{\cdot}8}\right\}$ but to allow for contingencies in casting it would be well to add to the thickness $\cdot 125\ (1 - t)$, or in other words, a $\frac{1}{64}$ inch for every $\frac{1}{8}$ inch the thickness falls short of 1 inch.

DIAMETER AND THICKNESS BY AUTHOR'S RULE.

Say $22\frac{1}{2}$ diameters long, $d = \frac{15 \times 12}{22{\cdot}5} = 8$ inches.

$a = {\cdot}24 \times 8^2 = 15{\cdot}36$ sq. inches, $W = 15{\cdot}36 \times 2 = 30{\cdot}72$ tons.

∴ Column = 8 inches diameter, with $\frac{3}{8}$ inches metal, or adding $\cdot 125\ (1 - t)$ for contingencies, $\cdot 125\ (1 - {\cdot}66) = {\cdot}04$, ${\cdot}66 + {\cdot}04 = {\cdot}7$

∴ say column 8 inches diameter, with $\frac{7}{10}$ inch metal.

SHIELDS' RULE FOR LOADING COLUMNS.

Ends flat and fixed, with base plate. From 20 to 24 diameters in length.

$\frac{3}{4}$ inch	thick	and upwards	= 2	tons	per sq. inch.
$\frac{5}{8}$	"	"	$= 1\frac{3}{4}$	"	"
$\frac{1}{2}$	"	"	$= 1\frac{1}{2}$	"	"
$\frac{3}{8}$	"	"	$= 1\frac{1}{4}$	"	"

DIAMETER AND THICKNESS BY SHIELDS' RULE.

30 tons on column 15 feet long, say length = 20 diameters, then $d = \frac{15 \times 12}{20} = 9$ inches, $t = \frac{1}{12}\ d = \frac{9}{12} = {\cdot}75$ inches,

$a = \cdot 24\ d^2$, $= \cdot 24 \times 9^2 = 19 \cdot 44$ sq. inches, W $= 19 \cdot 44 \times$ 2 tons $= 38 \cdot 88$ tons.

This is rather too much, so take $d = \frac{l}{22}$ then $d = \frac{15 \times 12}{22}$ $= 8 \cdot 18$ inches, $t = \frac{1}{12}\ d$, $\frac{8 \cdot 18}{12} = \cdot 682$ inches, $a = \cdot 24 \times 8 \cdot 18^2$ $= 16 \cdot 06$ sq. inches, W $= 16 \cdot 06 \times 1 \cdot 75 = 28 \cdot 1$ tons.

This is rather too little; assume $d = 8 \cdot 25$, then $t = \frac{8 \cdot 25}{12}$ $= \cdot 69$, say $\cdot 7$, $a = \cdot 24 \times 8 \cdot 25^2 = 16 \cdot 33$ sq. inches, W $= 16 \cdot 33$ $\times$ say $1 \cdot 875 = 30 \cdot 6$ tons, which is sufficiently near to the required load.

∴ Column $8\frac{1}{4}$ inches diameter, with $\frac{7}{10}$ inch metal.

DIAMETER AND THICKNESS BY ANOTHER RULE.

t = thickness in inches (not to exceed $\frac{1}{6}\ d$) d = diameter in inches (external, $\frac{1}{10}$ to $\frac{1}{20}$ length), L = length in feet, ends flat and fixed.

$$\text{Safe load tons per square inch} = (t + 1)\ \frac{2\,d}{\text{L}}.$$

By this rule a column 15 feet long, $8\frac{1}{4}$ inches diameter, $\frac{7}{10}$ metal would bear safely $32 \cdot 67$ tons, and one of the same length and thickness of metal but 8 inches diameter would bear $27 \cdot 72$ tons, or the equivalent for 30 tons would be, say

Column $8\frac{1}{8}$ inches diameter, with $\frac{7}{10}$ inch metal.

HODGKINSON'S FORMULA.

From Rankine's 'Civil Engineering,' p. 236.

For long columns (length = 30 diameters or over).

D = outside diameter inches.
d = inside „ „
L = length feet.

$$\text{B.W. tons} = \frac{44{\cdot}3\,(D^{3{\cdot}6} - d^{3{\cdot}6})}{L^{1{\cdot}7}} = C.$$

For short columns (length = less than 30 diameters).

s = sectional area sq. inches.
f = ultimate crushing strength, short block = 49 tons per sq. inch.

$$\text{B.W. tons} = \frac{C f s}{C + \frac{3}{4} f s}.$$

[NOTE.—Hodgkinson gave 44·34, 3·55, and 1·77; observe that 44·3 × 2240 = approximately 100,000.]

STRENGTH BY HODGKINSON'S FORMULA.

With this, as with the majority of formulæ for columns, the dimensions are first assumed and then tested for sufficiency.

Take column 15 feet long, 8 inches diameter, $\frac{7}{10}$ inch metal = 16·05 sq. inches sectional area. This will be 22½ diameters long, and will therefore come under the rule for short columns, *but the formula for short columns involves the use of that for long columns as a means to an end.*

$$\therefore\ C = \frac{44{\cdot}3\,(8^{3{\cdot}6} - 6{\cdot}6^{3{\cdot}6})}{15^{1{\cdot}7}}.$$

(See Fig. 92, Plate 8, for method of finding fractional powers without logarithms.)

By logarithms.

Number.		Index.	Mantissa.
8	=	0	9030900.

To raise a number to its power, multiply its log.

$$\therefore\ 0{\cdot}9030900 \times 3{\cdot}6 = 3{\cdot}2511240$$

log. ·2511270 = number 17829

and index 3, places decimal point thus, 1782·9, so that $8^{3{\cdot}6} =$ 1782·9, and $C = \frac{44{\cdot}3\,(1782{\cdot}9 - 891{\cdot}98)}{99{\cdot}852} = 395$, whence

Breaking weight by formula $\frac{C f s}{C + \frac{3}{4} f s}$

$$= \frac{395 \times 49 \times 16{\cdot}05}{395 + (\cdot 75 \times 49 \times 16{\cdot}05)} = 315{\cdot}5 \text{ tons,}$$

or a factor of safety of $\frac{315{\cdot}5}{30} = 10{\cdot}5$.

HURST'S TABLES.

The strength of hollow columns nearly equals the difference between that of two solid columns, the diameters of which are equal to the external and internal diameters of the hollow one.—Hurst's 'Handbook,' p. 113.

STRENGTH BY HURST'S TABLES.

8 inches $= \frac{80{\cdot}31 + 64}{2} = 72{\cdot}16$ tons, 6·6 inches $= \frac{6{\cdot}6}{6{\cdot}5} \times \frac{38{\cdot}03 + 30{\cdot}31}{2} = 34{\cdot}68$ tons.

$72{\cdot}16 - 34{\cdot}68 = 37{\cdot}48$ tons, as a long column, or

$$\frac{37{\cdot}48 \times 49 \times 16{\cdot}05}{(10 \times 37{\cdot}48) + (\cdot 75 \times 49 \times 16{\cdot}05)} = 30{\cdot}56 \text{ tons, safe load}$$

at $\frac{1}{10}$ breaking weight as a short column; or for a load of 30 tons, a factor of safety of $\frac{30 \cdot 56 \times 10}{30} = 10 \cdot 185$.

RONDELET'S FORMULA.

Breaking weight ends fixed $= f_c \, A \, k$ where $f_c = 42$ tons per sq. inch, A = area in sq. inches, k = constant varying with ratio $l \div d$.

$\frac{l}{d} =$	12	24	36	48	60	72
then $k =$	$\frac{20}{24}$	$\frac{12}{24}$	$\frac{7}{24}$	$\frac{4}{24}$	$\frac{2}{24}$	$\frac{1}{24}$

or approximately if $r = \frac{l}{d}$ then within usual limits $k = 1 - \frac{r}{50}$.

STRENGTH BY RONDELET'S FORMULA.

B. W. tons on column 15 feet long, 8 inches diameter, $\frac{7}{10}$ inch metal $= 42 \times 16 \cdot 05 \times \left(1 - \frac{22 \cdot 5}{\cdot 50}\right) = 370 \cdot 7$ tons, giving a factor of safety of $\frac{370 \cdot 7}{30} = 12 \cdot 36$.

GORDON'S FORMULA.

For columns fixed at both ends by having flat capitals and bases.

P = crushing load of a *long* rod or pillar in pounds.

f = 80,000 (ultimate resistance to crushing of a short block in pounds per sq. inch), or about 36 tons.

S = sectional area of material in sq. inches.

$a = \frac{1}{800}$ constant deduced from Hodgkinson's experiments.*

l = Length
h = Least external diameter } both in same units of measure,

then approximately

$$P = \frac{fS}{1 + a\frac{l^2}{h^2}}.$$

STRENGTH BY GORDON'S FORMULA.

$$\text{Crushing load, lbs.} = \frac{80{,}000 \times 16{\cdot}05}{1 + \frac{1}{800}\frac{(15 \times 12)^2}{8^2}} = 786{,}379 \text{ lbs.}$$

$$= 351{\cdot}06 \text{ tons, or a factor of safety of } \frac{351}{30} = 11{\cdot}7.$$

REULEAUX'S FORMULA.

$$\text{Breaking weight} = 4\pi^2\frac{IE}{l^2}.$$

For hollow circle (Rankine's 'Applied Mechanics,' p. 82)

$$I = \frac{\pi}{64}(D^4 - d^4).$$

* There is some doubt as to the value of a. Hurst ('Handbook,' p. 46) gives $a = \frac{1}{400}$ for "round hollow pillars, ends flat and fixed." Molesworth ('Pocket-Book,' p. 114) gives it in another form which equals $\frac{1}{400}$. Wray ('Theory of Construction,' p. 97), referring to hollow columns, gives $a = \frac{1}{400}$ for "cast-iron columns fixed at both ends." The 'Builder,' June 23, 1888, gives $a = \frac{1}{400}$ for "pillars, both ends flat, and carefully bedded." Trautwine ('Pocket-Book,' p. 221) gives $a = \frac{1}{600}$ for "hollow cast-iron cylindrical pillars." Rankine (Civil Engineering,' p. 237) gives $a = \frac{1}{800}$ for "cast-iron (hollow cylindrical pillars)." The latter is assumed to be correct.

For average cast iron (Rankine's 'Applied Mechanics,' p. 631)

$$E = 17{,}000{,}000.$$

$$\therefore \text{ B.W.} = \frac{4\pi^2 \frac{\pi}{64} (D^4 - d^4)\, 17{,}000{,}000}{l^2}$$

$$= \frac{32{,}800{,}000\,(D^4 - d^4)}{l^2},$$

or for our column of 8 inches diameter, $\frac{7}{10}$-inch metal, 15 feet high,

$$\text{breaking weight} = \frac{32{,}800{,}000 \times (4096 - 1897{\cdot}5)}{32{,}400 \times 2240}$$

$= 993{\cdot}5$ tons, or a factor of safety of $\frac{993{\cdot}5}{30} = 33{\cdot}12$.

UNWIN'S FORMULA.

(Unwin's 'Machine Design,' p. 63.)

E = modulus of direct elasticity of material, for cast iron $= 17{,}000{,}000$.

I = moment of inertia of the section; for annular section $= {\cdot}0491\,(D^4 - d^4)$

l = length in inches.

m = constant depending on mode in which ends are fixed; for column fixed both ends $= {\cdot}5$.

n = factor of safety $= \frac{1}{8}$ for cast iron.

$$\text{Working load in lbs.} = \frac{\pi^2}{m^2} \times \frac{n\,E\,I}{l^2}$$

$$= \frac{3{\cdot}1416^2}{{\cdot}5^2} \times \frac{\frac{1}{8} \times 17{,}000{,}000 \times {\cdot}0491\,(8^4 - 6{\cdot}6^4)}{180^2} =$$

372,668·3 lbs. = ... 166·4 tons,

or more than 5 times the given load, giving a factor of safety of 33·3.

This is practically the same formula as Reuleaux's, both being founded upon Euler.

STRENGTH BY TIMMINS' TABLES.

8″ diameter, ¾″ thick, 12 ft. long, will carry 34 tons.

„	„	14	„	„	29	„
„	„	16	„	„	25	„
„	„	18	„	„	22	„

Plotting these loads to a curve, Fig. 116, it will be found that 15 feet long will carry about 27 tons, and allowing for the difference in sectional area between ¾ inch thick (17·08) and $\frac{7}{10}$ inch thick (16·05)

$$17{\cdot}08 \;:\; 16{\cdot}05 \;::\; 27 \;:\; 25{\cdot}37 \text{ tons,}$$

and as the tables are said to be calculated for $\frac{1}{10}$ of breaking weight, the factor of safety will be $\frac{25{\cdot}37}{30} = 8{\cdot}46$.

STRENGTH BY WYBORN'S TABLES.

8″ diameter, ¾″ thick, 10 ft. long, will carry 39·3 tons.

„	„	12	„	„	34·0	„
„	„	16	„	„	25·2	„
„	„	20	„	„	18·7	„

Plotting these to a curve, the same result is obtained as before.

SUMMARY OF RESULTS.

By author's rule, column 8 in. diameter, $\frac{7}{10}$ in. metal.
„ Shields' „ „ $8\frac{1}{4}$ „ $\frac{7}{10}$ „
„ another „ „ $8\frac{1}{8}$ „ $\frac{7}{10}$ „

Assuming 8 inches diameter and $\frac{7}{10}$ inch metal, then

By Hodgkinson's formula,	factor of safety	=	10·5
„ Hurst's Tables	„	=	10·185
„ Rondelet's formula	„	=	12·36
„ Gordon's formula	„	=	11·7
„ Reuleaux's formula	„	=	33·12
„ Unwin's formula	„	=	33·3
„ Timmins' Tables	„	=	8·46
„ Wyborn's Tables	„	=	8·46

The comparison is not quite satisfactory, but appears to show that in using a cast-iron hollow column 15 feet long, 8 inches external diameter, $\frac{7}{10}$ inch thickness of metal to carry a dead load of 30 tons, we shall have a reliable factor of safety of 10. See remarks on Stanchions, pp. 112 and 113.

OUTLINE OF COLUMNS.

Columns may be parallel, i. e. cylindrical, or tapered, i. e. conical, the outline being straight or swelled.

In common work, columns up to 9 inches diameter are frequently made parallel, with a square cap or abacus at each end about 2 inches larger than the diameter, and a large plain torus bead adjoining, see Fig. 117.

An improvement on this is shown in Fig. 118. The cap has raised sides to form a clip under girder, and is lengthened

in two directions to give more bearing for a wood beam. The fillet and hollow under torus moulding are also better in appearance than the plain bead.

The majority of plain columns are tapered, the bottom diameter being from $\frac{1}{8}$ to $\frac{1}{4}$ more than the top. The smallest diameter must be that upon which the strength of the column is calculated.

More rarely an entasis or swelling is given to the shaft of a long column to prevent the sides looking hollow, see Figs. 119 and 120. Hodgkinson's experiments proved that the entasis does not add to the strength of a hollow column fixed at the ends.

Ornamental columns must first be designed as plain, and then clothed with whatever ornament may be required, avoiding abrupt changes of thickness and providing for facility and economy of manufacture.

In this case $\frac{1}{8}$ diameter = 1 inch $\therefore$ bottom diameter = 9 inches.

BEDDING OF COLUMNS.

It is most important that cast-iron columns should be carefully bedded to distribute the pressure equally and avoid fractures. The under surface of a bed plate is always more or less irregular, and it is difficult to chip a casting even, owing to the hard skin. When resting on stone, the stone should be first dressed to the inequalities of the casting and then a sheet of lead between two layers of tarred felt inserted under the column, or a thickness of tarred felt only; or cement grout or melted sulphur and sand may be run in between the iron and the stone.

Sixty tons of iron columns fixed by the author in 1884 on

concrete beds 6 feet square × 16 feet deep, were supported by wedges ¾ inch above the top of concrete, and a clay bank being put round, cement grout was run in. Many of these columns carry a working load of 120 tons each, and show no signs of failure.

At the Broad Street Goods Station of the London and North Western Railway Company, a row of columns on expensive granite bases at the yard level supported the girders carrying the permanent way at the railway level. Owing to the cracking of several of these bases upon completion of the work, two additional similar columns and bases were put at the side of each of the original columns.

STONE BASE.

A piece of tooled York stone is commonly used, called a piece of *ashlar*, the dimensions of which may be calculated according to the notes at p. 43. The minimum area should be twice that of bottom flange of column.

L. de C. Berg says, "In stonework with level dressed beds allow for load $\frac{4}{10}$ of strength of stone." This would fairly represent the practice of many architects, but it gives much too low a factor of safety. When, however, the column is fairly placed in the centre of the stone, the latter does gain some strength from the margin round the base, as indicated in the following paragraph, which shows that *half* the load to crush the whole area of stone will produce crushing if concentrated on the central *quarter* of the area, or in other words, the intensity of pressure may be doubled if limited to the central quarter of the area.

STRENGTH OF STONE

Varies according to comparative area of loading.

Crushing load uniformly distributed over whole surface	= W
Do. limited to central half of the area .	= ·7 W
Do. limited to central quarter of the area	= ·5 W
Or the pressure per unit of area under load when whole surface loaded .	= *w*
Do. when central half of area only is loaded	= 1·4 *w*
Do. when central quarter of area only is loaded	= 2 *w*

—*Unwin.*

BOTTOM FLANGE.

This may be proportioned as described for bearing area at p. 43, when the column carries a variable load, but when it carries a fixed dead load, the weight per foot super. may be increased to a maximum of one-third to one-half more. Approximately the side of a square flange may be made equal to twice the diameter of the column at the base. In this case 12 tons per sq. foot safe load would become 16 tons, and 16 tons : 144 sq. inches : : 30 tons : 270 sq. inches, $\sqrt{270}$ = 16·4 inches side, while bottom diameter × 2 = 9 × 2 = 18 inches side.

There is usually a hole for the core bar in the bottom of a column, varying from 2 inches diameter up to the full inside diameter of column. An addition should be made to the flange area to allow for this in important cases. The same

may occur in the top of column but sometimes the hole there is square.

The flanges of columns are frequently made too small, especially those that may be bought ready made.

TOP FLANGE.

This will vary considerably, according to the purpose for which the column is intended; some forms are shown in Figs. 121 to 125.

When the column carries a baulk of timber, or a wood girder, the top flange is made long to give sufficient bearing surface, and sometimes stiffened by the edges being raised, as in Fig. 118.

SHORTENING UNDER COMPRESSION.

Trautwine* says, cast-iron columns shorten at the average rate of about $\frac{1}{4}$ in 30 feet, under loads of 4 tons per sq. inch of metal cross-section.

Assuming the compression to be proportionate to length and load per sq. inch, our column under full load will be shortened

$$\frac{2 \times 15 \times \frac{1}{4}}{4 \times 30} = \frac{1}{16} \text{ inch.}$$

WEIGHT OF COLUMNS.

The estimated weight may be arrived at by measurement from the drawings, allowing cast iron to weigh 450 lbs. per cubic foot; take care to allow for fillets in all angles, and err on the side of fulness rather than otherwise.

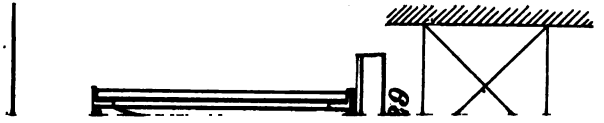
In settling up a contract the original invoices of the

* 'Pocket-Book,' p. 221.

columns should be produced, and the excess or deficiency of weight upon the quantities, as the case may be, allowed for. If any suspicion be entertained as to the accuracy of the invoices an occasional column must be carted back and re-weighed.

CASTING OF COLUMNS.

All columns should be cast *vertically*, with a head for sullage, to secure close grained metal in the body of the casting. When this is specified an angle of 30° from the horizontal is generally claimed by the founder as fulfilling the contract, but casting at this angle only partially attains the desired object.



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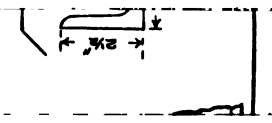
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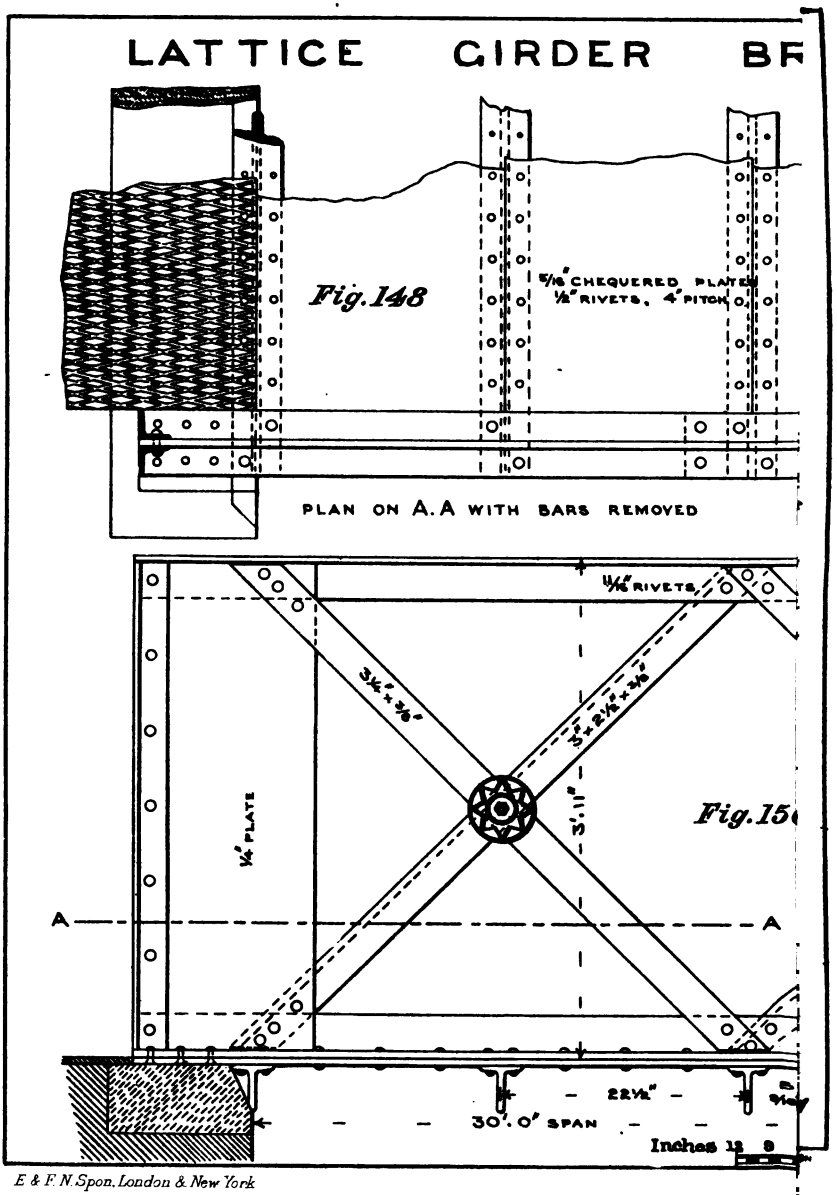

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CHAPTER X.

LATTICE GIRDER BRIDGE, 30 FEET SPAN, BETWEEN TWO WAREHOUSES.

(See Plates 11 and 12.)

THE bridge is to be suitable for trucking loaded sacks across from one warehouse to the other, to have a clear width of 6 feet between the flanges, to be sufficiently strong to carry a dead load of 2½ cwt. per foot super., in addition to its own weight, and to be constructed entirely of wrought iron.

The general arrangement proposed to be adopted should be sketched out to guide the calculations, remembering that every detail is subject to modification in the after stages if required. Sometimes even the principal feature of the design has to be altered when brought to the test of calculation, in consequence of a misapprehension of its suitability. Architects are too apt to elaborate the design of ironwork before giving sufficient attention to the stresses, while engineers do the reverse and neglect appearance for the sake of simplicity in the parts; no work can be perfectly satisfactory unless design and calculation go hand in hand.

APPROXIMATE DESIGN.

Fig. 128 shows a rough sketch of part of proposed elevation, and Fig. 129 a section, the floor being formed of L iron, T iron, or deck beams, covered with wrought-iron chequered plate.

USUAL ALLOWANCE FOR LOAD ON FLOORS.

Churches and Public buildings	1½ cwt. per sq. foot.
Warehouses . . .	2½ „ „
Dwelling houses . . .	1¼ „ „

By the statement of requirements, 2½ cwt. per foot super. has to be allowed for, in addition to the weight of the bridge. It is assumed that this will be ample for the effect of as many moving loads of 2½ cwt. in sack barrows as can at any time be upon, or passing over, the bridge. We must look into this. By actual measurement the ground space occupied is as follows:

		Cwt. per ft. super.
2½ cwt. sack of flour . .	2·625 feet super.	= ·952
„ „ grain . .	2·00 „	= 1·250
2 cwt. „ coal . .	1·58 „	= 1·266

therefore merchandise in sacks, one tier deep, cannot overload the bridge, as the maximum resulting weight is only 1·27 cwt. per foot super.

A porter's hand-barrow or sack barrow weighs on the average ¾ cwt., and with a 2½ cwt. full sack upon it, occupies 7 sq. feet of ground space, giving a resulting weight of ·464 cwt. per foot super. The weight in this case is really supported over a very small area, which we shall have to consider in getting out detail of floor; but the size of the barrow and sack preventing the nearer approach of anything else, it is right now to take the weight as being distributed over the whole of the area.

If the goods be in transit over the bridge there might be, on extraordinary occasions, 3 rows, with minimum distance of

4 feet 6 inches from man to man in each row. This would give a maximum of $\frac{30}{4 \cdot 5} = 6 \cdot 66$, $6 \cdot 66 \times 3 = 20$ men and barrows. Each man would weigh on the average 10 stone = $1\frac{1}{4}$ cwt. Then $2\frac{1}{2} + \frac{3}{4} + 1\frac{1}{4} = 4\frac{1}{2}$ cwt. spread over $\frac{30 \times 6}{20} =$ 9 sq. feet, or $\frac{4 \cdot 5}{9} = \cdot 5$ cwt. per foot super. A moving load, however, may be assumed to strain the girder in the proportion of 8 to 5 over a dead load, which will bring the effect up to $\cdot 5 \times \frac{8}{5} = \cdot 8$ cwt. per foot super., still far below the allowance which has been given.

Another mode by which the bridge may be loaded is the collection of a crowd of men upon it.

WEIGHT OF MEN IN CROWDS.

Authority.	lbs. per sq. foot.
Cowper	140
Parsey	80 to 112
Young	80
Page	84
Usual practice	100

and taking the average weight of men at 100 lbs. per foot super., we have ·889 cwt. per foot super. over the bridge.

SUMMARY OF MODES OF LOADING.

Full sacks, one tier deep.	1·27 cwt. per foot super.
Sack-barrows and full sacks	·464 „ „
„ „ and man	·5 „ „
Men in crowd	·889 „ „

So that with an allowance of $2\frac{1}{2}$ cwt. per foot super. we are safe against all ordinary contingencies.

TOTAL NET LOAD ON BRIDGE.

$$\frac{30 \text{ ft. span} \times 6 \text{ ft. wide} \times 2\frac{1}{2} \text{ cwt. per ft. sup.}}{20 \text{ cwt. in } 1 \text{ ton}} = 22{\cdot}5 \text{ tons}$$

external dead load distributed over two girders.

WEIGHT OF STRUCTURE.

The approximate weight of such a structure may be taken as $\frac{W\,l}{180}$, where W = total external distributed load to be provided for, and l = span in feet. In this case $\frac{22{\cdot}5 \times 30}{180}$ = ... 3·75 tons.

Draughtsmen should keep a record of every structure designed by them, showing the relation between the span, area, external load and structural load, together with any details of cost, &c., that may be available. It is only by these means that they can gain such experience as will enable them to produce approximate estimates with reasonable rapidity.

GROSS LOAD ON EACH GIRDER.

$\frac{22{\cdot}5 + 3{\cdot}75}{2}$ = . . . 13·125 tons distributed.

DEPTH OF GIRDER.

Approximately $\frac{1}{8}$ to $\frac{1}{12}$ of the span, but will be decided by the number of bays formed by the lattice bars, and the height which may be most convenient. It will clearly be an advantage to make the main girders deep to act as barriers

or handrails, and we should therefore select 8 bays, unless it be found desirable to have an odd number to give a central one, when we should take 9 as the number. The number of raking struts (marked *a* in Fig. 129) provided on each, to counteract the lateral weakness of such girders, will guide us in settling this point. If fixed 3 bays apart throughout, a girder of 9 bays will be required to give a symmetrical appearance; 2 bays apart would be suited by a girder of 8 bays; but the girders being supported firmly against outward pressure by the walls at each end, the struts need not be so close together at the ends as near the centre. If we place them 2 bays apart in the centre, 3 bays will be left at each end, with an 8-bay girder, and this would probably give the best result (see Fig. 130). The girder being 8 bays long, and the lattice bars being always placed at an angle of 45°, the depth of the girder measuring from the intersections of the bars above and below (see *d*, Fig. 128) will be $\frac{1}{8}$ of the length between similar points at each end.

These points or intersections of end bars with bottom flange should, as a rule, come exactly over the edge of bearing surface, the chamfer on each stone being added to the clear span to obtain the effective span. The portion of lattice girder over bearing surface at each end should have a vertical web-plate inserted to stiffen it, the whole of this part being known as the end pillar.

CHAMFER.

On edge of bearing surface to prevent chipping.

$$\frac{W + \sqrt{s}}{10} = \frac{13{\cdot}125 + \sqrt{30}}{10} = 1{\cdot}8, \text{ say} \quad . \quad . \quad 2 \text{ inches.}$$

The two chamfers, adding 4 inches to the span of the girders, will increase the actual load; but, as the addition is only about 1 per cent., it need not be considered.

STRESSES IN LATTICE GIRDERS.

The simplest way to obtain these is by a "reciprocal diagram." Rules for constructing reciprocal diagrams will be found in the author's 'Strains in Ironwork,' p. 56. Fig. 131 shows the mode of numbering the outline of girder, and allotting the subdivision of the load.

$$\frac{13 \cdot 125 \text{ tons load}}{8 \text{ bays}} = 1 \cdot 64 \text{ tons at each apex.}$$

$\frac{1 \cdot 64}{2} = \cdot 82$ tons at each end, resting direct upon abutment without straining girder.

Fig. 132 shows the corresponding reciprocal diagram, the measurement of the lines giving the stresses in each girder, which are marked upon the various parts in Fig. 133. The lettering of loop at abutment is explained at p. 62, and Fig. 158, of 'Strains in Ironwork,' and the position of point A is found as described in p. 63, Fig. 163, of the same work, which may also be proved by commencing the diagram at further end, and working backwards.

Or the stresses might have been calculated, as shown by Fig. 7, in terms of x, x being the load on each apex = 1·64 tons.

ARRANGEMENT OF JOINTS.

This stage is the most difficult in designing lattice girders, and upon it the efficiency of the design depends. An adequate

number of rivets must be allowed to transmit the stress through the various parts, and substance must be provided in the parts to take the rivets without involving undue sectional area throughout the remainder of their lengths. The number of rivets required will affect the dimensions and form of section of the bars. As a principle to keep in view, always begin the junction with the fewest rivets possible, so as not to weaken the bar until a firm connection is obtained. (See Figs. 31, 32, 33, 137, 138.)

RIVETS.

Nominal Diameter.	Practical Diameter.	Theoretical Area.	Practical Area.	Safe Strength Tons. Single Shear.
$\frac{1}{2}$	·55	·1963	·24	1·20
$\frac{9}{16}$	·61	·2485	·29	1·45
$\frac{5}{8}$	·67	·3068	·35	1·75
$\frac{11}{16}$	·74	·3712	·43	2·15
$\frac{3}{4}$	·80	·4417	·50	2·50
$\frac{13}{16}$	·86	·5185	·58	2·90
$\frac{7}{8}$	·92	·6013	·66	3·30
$\frac{15}{16}$	·99	·6903	·77	3·85
1	1·05	·7854	·87	4·35

Ultimate shearing strength of a Low Moor rivet in tons $= 18\ d^2$, working strength $= 4{\cdot}5\ d^2$ (d = actual diameter). Mean tensile strength of rivet iron in tons per sq. inch, Fair-

bairn 26·3, Kirkaldy 25·98. See Chap. III., pp. 28 and 32, for further information.

Assuming $\frac{5}{8}$ inch rivets, the safe shearing stress on each will be 1·75 tons, and the maximum stress in diagonal bars of 4·64 tons will require $\frac{4 \cdot 64}{1 \cdot 75} = 2 \cdot 6$, say 3 rivets. But n rivets placed singly require a distance or length of

$$(n \times 2\tfrac{1}{2}\, d) + \tfrac{1}{2}\, d$$

and therefore 3 rivets would require

$$(3 \times 2\tfrac{1}{2} \times \tfrac{5}{8}) + (\tfrac{1}{2} \times \tfrac{5}{8}) = (3 \times 2 \cdot 5 \times \cdot 625) + \frac{\cdot 625}{2} = 5 \text{ in.}$$

for the depth of web of T iron, or placed thus ⁂ would require a width of bar, for the two rivets in line, of

$$(2 \times 2 \cdot 5 \times \cdot 625) + \frac{\cdot 625}{2} = 3 \cdot 4375 = 3\tfrac{7}{16} \text{ inches.}$$

It is probable that by slightly increasing the diameter of the rivets two would be sufficient, and a saving would result in the dimensions of the parts through which they pass.

Assume $\frac{11}{16}$-inch rivets, then the safe shearing strength will be 2·15 tons, and $\frac{4 \cdot 64}{2 \cdot 15} = 2 \cdot 1$, or say two rivets, requiring a depth of web of under 4 inches, or a width of bar of say 2 inches.

BARS UNDER TENSION.

These will be ties, as shown by thin lines on Fig. 133, safe load 5 tons per sq. inch net area.

FIRST BAR, $\frac{4 \cdot 64}{5} = \cdot 93$ sq. inches net area.

Say $\frac{3}{8}$ inch or ·375 inch thick, then $\frac{\cdot 93}{\cdot 375} = 2 \cdot 48$ inches net width.

$2 \cdot 48 + \cdot 74$ (for one $\frac{11}{16}$-inch rivet) $= 3 \cdot 22$ inches gross width,

say . . . . . $3\frac{1}{4}'' \times \frac{3}{8}''$.

NEXT BAR, $\frac{3 \cdot 48}{5} = \cdot 696$, $\frac{\cdot 696}{\cdot 375} = 1 \cdot 86$, $1 \cdot 86 + \cdot 74 = 2 \cdot 6$,

say . . . . . $2\frac{3}{4}'' \times \frac{3}{8}''$.

NEXT BAR, $\frac{2 \cdot 32}{5} = \cdot 464$, $\frac{\cdot 464}{\cdot 375} = 1 \cdot 24$, $1 \cdot 24 + \cdot 74 = 1 \cdot 98$.

In this and the remaining bar, the stress being so much reduced, the diameter of the rivets may be reduced: thus $\frac{2 \cdot 32}{2} = 1 \cdot 16$ shearing stress on each rivet, which would be met by using $\frac{1}{2}$-inch rivets; but $\frac{1}{2}$ inch is a very likely thickness for the T iron, and it is difficult to punch $\frac{1}{2}$-inch holes in that, so we may say $\frac{9}{16}$-inch rivets, having a practical diameter of ·61, then $1 \cdot 24 + \cdot 61 = 1 \cdot 85$,

say . . . . . $2'' \times \frac{3}{8}''$.

NEXT BAR, $\frac{1 \cdot 16}{5} = \cdot 232$, $\frac{\cdot 232}{\cdot 375} = \cdot 62$, $\cdot 62 + \cdot 61 = 1 \cdot 23$.

$\therefore$ $1\frac{1}{4}$ inch wide would apparently be sufficient, but there is a possibility of bursting the hole in punching if the bar is much narrower than three times the diameter of the rivet,

say . . . . . $1\frac{1}{2}'' \times \frac{3}{8}''$.

BARS UNDER COMPRESSION.

These will be struts, as shown by thick lines in Fig. 133.

W = working load in tons;

l = unsupported length in inches;

d = least depth or width in inches ;

a = area in square inches allowing a factor of safety of eight (rivets not deducted).

$$a = \frac{W}{2} + \cdot 0003\, W \left(\frac{l}{d}\right)^2.$$

The bars will be connected where they cross each other, making the unsupported length of bar =

$$\frac{\frac{1}{8}\text{th of 30 feet span} \times \sqrt{2} \times 12}{2} = 31 \cdot 815, \text{ say 32 inches.}$$

$$\text{FIRST BAR, } \frac{3 \cdot 48}{2} + \left(\cdot 0003 \times 3 \cdot 48 \frac{32^2}{d^2}\right) = 1 \cdot 74 + \frac{1 \cdot 068}{d^2},$$

assuming $d = 3$ inches for side of angle iron, then

$$1 \cdot 74 + \frac{1 \cdot 068}{3^2} = 1 \cdot 86 \text{ sq. inches area.}$$

The table given on the next page will be found useful for determining the dimensions to provide a given area.

We require 1·86 sq. inches area with a 3-inch width of angle iron, narrowest side, we must therefore take $3 \times 3 \times \frac{3}{8} = 2 \cdot 12$, unless we make one side less than 3 inches, when we must go through the calculation again. Say $3 \times 2\frac{1}{2}$, then

$$1 \cdot 74 + \frac{1 \cdot 068}{2 \cdot 5^2} = 1 \cdot 91 \text{ square inches area,}$$

and by reference to the table this would require

say . . . . . $3'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$.

$$\text{NEXT BAR, } \frac{2 \cdot 32}{2} + \left(\cdot 0003 \times 2 \cdot 32 \frac{32^2}{d^2}\right) = 1 \cdot 16 + \frac{\cdot 7127}{d^2},$$

say side of angle iron 2 inches, then

$$1 \cdot 16 + \frac{\cdot 7127}{2^2} = 1 \cdot 34 \text{ square inches area,}$$

say . . . . . $2\frac{1}{2}'' \times 2'' \times \frac{5}{16}''$.

AREA OF ANGLE OR TEE IRON IN SQUARE INCHES.

(See Fig. 139.)

Size of Angle or Tee. Width and Depth.	Thickness.							
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{3}{4}$
$1\frac{1}{2}\times1\frac{1}{2}$, 2×1 .	·69							
$1\frac{3}{4}\times1\frac{3}{4}$, $2\times1\frac{1}{2}$.	·81	1·00						
2×2, $2\frac{1}{2}\times1\frac{1}{2}$.	·94	1·16	1·37					
$2\frac{1}{4}\times2\frac{1}{4}$, $2\frac{1}{2}\times2$, $3\times1\frac{1}{2}$	1·06	1·32	1·50					
$2\frac{1}{2}\times2\frac{1}{2}$, 3×2, $3\frac{1}{2}\times1\frac{1}{2}$	1·19	1·47	1·74	2·00	2·26			
$2\frac{3}{4}\times2\frac{3}{4}$, $3\times2\frac{1}{2}$.	..	1·63	1·93	2·23	2·51			
3×3, $3\frac{1}{2}\times2\frac{1}{2}$, 4×2	..	1·78	2·12	2·44	2·77	3·08		
$3\frac{1}{4}\times3\frac{1}{4}$, $3\frac{1}{2}\times3$, $4\times2\frac{1}{2}$	..	1·94	2·31	2·67	3·02	3·36	3·69	
$3\frac{1}{2}\times3\frac{1}{2}$, 4×3 .	..	2·10	2·50	2·88	3·28	3·65	4·01	
$3\frac{3}{4}\times3\frac{3}{4}$, $4\times3\frac{1}{2}$, $4\frac{1}{2}\times3$	..	..	2·69	3·11	3·52	3·92	4·32	
4×4, $4\frac{1}{2}\times3\frac{1}{2}$, 5×3	..	..	2·86	3·32	3·77	4·22	4·64	
$4\frac{1}{2}\times4\frac{1}{2}$, 5×4, 6×3	..	..	3·25	3·77	4·28	4·78	5·27	6·22
5×5, 6×4 . .	..	..	..	..	4·77	5·34	5·89	6·98
6×5 . . .	..	..	..	..	5·28	5·91	6·52	7·73
6×6, 8×4 . .	..	..	..	..	5·78	6·47	7·15	8·48

Area sq. inches $\times 3\frac{1}{3}$ = weight per foot run in lbs.

NEXT BAR, $\frac{1\cdot16}{2} + \left(\cdot0003 \times 1\cdot16\frac{32^2}{d^2}\right) = \cdot58 + \frac{\cdot3563}{d^2}$,

say side of angle iron $1\frac{1}{2}$ inch, then

$$\cdot58 + \frac{\cdot3563}{1\cdot5^2} = \cdot738 \text{ sq. inches area,}$$

say . . . . . 2″ × 1½″ × ¼″.

NEXT BAR, stress nil, make bar same as last,

say . . . . . 2″ × 1½″ × ¼″.

In the case of a single small bridge of this kind it would not pay to have so many different sizes of angle iron and bars, it would be better to make them all uniform, taking the maximum as the standard, viz.

Tension bars 3¼″ × ⅜″, Compression bars 3″ × 2½″ × ⅜″.

We may, however, assume that there will be a similar bridge on each floor at each loophole of the warehouse, and make all the parts proportional to the stresses, for the sake of the practice it gives.

TOP FLANGE.

When the stress is small, a T iron is the best section for the top and bottom flanges, as, being solid, there is a saving in labour, and corrosion has less effect. If made up of angle irons, the space between cannot be reached by the painter's brush, and oxidation is likely to occur, unless the space in bottom flange be filled up with Portland cement. In larger girders a dwarf web plate between angle irons makes a good flange and gives plenty of room for rivets.

The stress varies here from 13·12 tons to 3·28 tons; 13·12 at 4 tons per sq. inch compression, not deducting rivet holes, = $\frac{13\cdot12}{4} = 3\cdot28$ sq. inches, giving say a 4″ × 3″ × ½″ tee iron;

but the portion of flange between each apex is under the condition of a strut, and must therefore be calculated as such.

Assume 3 inches as smallest dimension, then

$$\frac{W}{2} + \cdot 0003\, W \left(\frac{l}{d}\right)^2$$

$$= \frac{13 \cdot 12}{2} + \left\{ \cdot 0003 \times 13 \cdot 12 \times \frac{(3 \cdot 75 \times 12)^2}{d^2} \right\}$$

$$= 6 \cdot 56 + \frac{7 \cdot 97}{d^2} = 6 \cdot 56 + \frac{7 \cdot 97}{3^2} = 7 \cdot 45 \text{ sq. inches.}$$

But on reference to the table we cannot get this area with a tee iron, having 3 inches for one dimension. Try 4 inches for least width, then

$$6 \cdot 56 + \frac{7 \cdot 97}{4^2} = 7 \cdot 06 \text{ sq. inches,}$$

or say a tee iron 6″ × 4″ × ¾″. This would be very heavy, and extravagant of material towards the ends. We may, instead, take a lighter tee iron and rivet a stiffening plate to it as a flange plate, see Fig. 140, reaching over the four central bays.

Take the stress beyond this additional plate to obtain section of tee iron.

$$\frac{8 \cdot 2}{2} + \left\{ \cdot 0003 \times 8 \cdot 2 \frac{(3 \cdot 75 \times 12)^2}{d^2} \right\}$$

$$= 4 \cdot 1 + \frac{4 \cdot 98}{d^2} = 4 \cdot 1 + \frac{4 \cdot 98}{4^2} = 4 \cdot 4 \text{ sq. inches,}$$

say . . . . . 6″ × 4″ × ½″

and then for the plate on top,

$7 \cdot 06 - 4 \cdot 4 = 2 \cdot 66$ sq. inches area to make up.

$$\frac{2 \cdot 66}{6 \text{ inches wide}} = \cdot 44 \text{ inches thick,}$$

say . . . . . 6″ × ½″.

FLOOR PLATES.

Before we can estimate the bottom flange we must work out the floor plates and cross beams, in order to obtain the amount and position of the loads producing transverse stress upon this flange.

We propose to carry wrought-iron chequered floor plates upon T irons or deck beams, as shown in Fig. 128, viz. two to each bay, or $\frac{30}{16} = 1{\cdot}875$ feet centre to centre, but we overlooked the fixing of the stiffeners, *a*, Fig. 129, which ought to appear symmetrical in the elevation, and must come in line with the cross beams. We had better therefore shift the cross beams to come under each apex and centre of each bay, where they will be the same distance apart, see Fig. 141.

Common thicknesses for these plates are $\frac{1}{4}$ inch, $\frac{5}{16}$ inch, and $\frac{3}{8}$ inch, the weight per foot super being about 2 lbs. less than solid plate of same thickness, but we must calculate the thickness necessary to support the load. Each plate will be 6 feet long × 22½ inches (1·875 feet) wide, being limited by the positions of the lattice girders and the cross beams. With the distributed load of 2½ cwt. per foot super we shall have $1{\cdot}875 \times 2\frac{1}{2} \times 112 = 525$ lbs. per foot in width, with a span of 22·5 inches, and for solid plates, riveted along two edges,

$$t = \sqrt{\frac{{\cdot}000044\,\mathrm{W}\,l}{b}}$$

where t = thickness in inches, W = distributed load in lbs., l = span in inches, or shortest distance between supports, b = breadth in inches.

$$\therefore \quad t = \sqrt{\frac{{\cdot}000044 \times 525 \times 22{\cdot}5}{12}} = {\cdot}208, \text{ say } \tfrac{1}{4} \text{ inch thick.}$$

Before deciding, we must see what the effect of a concentrated load would be, such as one wheel of a loaded truck.

$$\frac{\text{Full sack } 2{\cdot}5 \text{ cwt.} + \text{truck } {\cdot}75 \text{ cwt.}}{2} \times 112 = 182 \text{ lbs.},$$

equivalent to $182 \times 2 = 364$ lbs. distributed, as against 525 lbs. provided for, but it is not usual to adopt chequered plates of less than $\frac{5}{16}$-inch thick, unless they have a wood backing,

∴ say . . . . . $6' \times 22\frac{1}{2}'' \times \frac{5}{16}''$.

NOTE.—Iron floor plates have been taken to show the mode of calculation, but probably the noise of the traffic over them would be objectionable, and a wood floor would be a desirable substitute unless they are specially required to be fireproof.

CROSS BEAMS.

For light loads, angle, tee, or channel irons may be used; for heavier loads, bridge or flange rails inverted, or deck beams; and for the heaviest loads rolled joists will be the most economical. The load in this case will be

External load $= 6 \times 1{\cdot}875 \times 2{\cdot}5 \times 112 = 3150$ lbs.

Weight of floor plate $= 6 \times 1{\cdot}875 \times (\frac{5}{16}$ of $40^* - 2)$ 118 lbs.

Weight of cross beam $= 6 \times$ say 12 lbs. per foot run $= 72$ lbs.

$3150 + 118 + 72 = 3340$ lbs. distributed.

On reference to 'Molesworth's Pocket Book,' p. 140, we find that the largest T there given, $4 \times 4 \times \frac{1}{2}$, only equals 14750, while 3340×6 feet span $= 20040$, but $K_1 = K \frac{D_1^3}{D^3}$ and $20040 = 14750 \frac{D^3}{4^3}$ ∴ depth required $= \sqrt[3]{\frac{20040 \times 4^3}{14750}} = 4{\cdot}43$, or say

* 40 = weight in lbs. of 1 foot super. 1 inch thick.

$4\frac{1}{2}$ inches. The rivets to hold floor plates down being in top flange will be in compression, so that no deduction will be required, and the whole bar will then be,

say . . . . . $4'' \times 4\frac{1}{2}'' \times \frac{1}{2}''$.

BOTTOM FLANGE.

Maximum tensile stress = 12·3 tons, which at 5 tons per sq. inch will require $\frac{12 \cdot 3}{5} = 2 \cdot 46$ sq. inches; but in this case we have also a cross strain to allow for, due to the beam supporting the floor and its load at centre of each bay = 3340 lbs., equivalent to $3340 \times 2 = 6680$ lbs. distributed.

The flange is, however, *continuous* and the stress will therefore only be half that due to a simply supported beam, or the equivalent load will be $\frac{6680}{2} = 3340$ lbs. distributed on a tee beam 3·75 feet span, supported at ends; and by Molesworth, p.140, $3340 \times 3 \cdot 75 = 12525$, which is beyond the range of the table there given for ⊥, the highest value being 11800 for $4 \times 4 \times \frac{1}{2}$ $\therefore$ $D = \sqrt[3]{\frac{12525 \times 4^3}{11800}} = 4 \cdot 08$ required depth, which is so little over 4 inches that we may say $4 \times 4 \times \frac{1}{2}$ tee iron to take the cross strain.

We have now to allow for the tensile stress which requires 2·46 sq. inches net. If we increase the tee iron to $6 \times 4 \times \frac{1}{2}$ we gain $2 \times \frac{1}{2} = 1$ sq. inch, and by adding a plate $6 \times \frac{1}{2}$ underneath we gain 3 sq. inches more, or 4 in all, from which we have to deduct rivet holes, say

1—	$\frac{9}{16}$-inch hole in	$\frac{1}{2}$-inch web	= ·31	sq. inches.
2—	„	„ flange	= ·62	„
2—	„	„ plate	= ·62	„

·31 + ·62 + ·62 = 1·55, and 4 − 1·55 = 2·45 sq. inches, or apparently not quite enough, but when we consider the extra stiffness to resist cross strain given by the increased dimensions we shall find we have ample section, because we may assume the part resisting the tensile stress to be situated about the neutral axis of the beam.

The additional plate will require to run through the adjoining bays, with a tension of 10·66 tons, $\frac{10 \cdot 66}{5} = 2 \cdot 13$ sq. inches, for which we are giving 2·45 sq. inches.

In the next bay, if we omit the extra plate, we shall have only the additional 2-inch width of tee iron = 1 sq. inch area and deducting rivet holes ·93 sq. inches we have practically nothing left.

It will therefore be necessary to run the extra plate through not less than six bays of the bottom flange, in order to provide sufficient material, making the flange of

tee iron	6″ × 4″ × ½″,
extra plate	6″ × ½″.

RIVETS IN FLOOR PLATES.

The floor plates must be riveted down to prevent displacement, and as the beams are ½-inch thick, we cannot possibly have the rivets less than ½-inch diameter, unless the holes are drilled. As the flange of the tee iron is in compression, the rivets may be opposite each other, thus, : : :, instead of zigzag. They should be "knock-down" rivets in countersunk holes to avoid causing obstruction to the trucks passing over bridge, say . . . . . ½-inch diam. × 4-inch pitch.

In ordinary foot-bridges it is usual to put diagonal "wind-

bracing" horizontally under the floor between the side girders, but in this case the construction of the floor with plates riveted to cross beams renders it exceptionally stiff, and diagonal bracing is not required.

RIVETS TO HOLD UP CROSS BEAMS.

We have assumed these to be $\frac{9}{16}$ inch diameter, but we must prove whether they will be sufficient. The load is 3340 lbs., or say $\frac{3340}{2} = 1670$ at each end. Area of $\frac{9}{16}$ inch rivets $= \cdot 3$ sq. inches, safe load $= 5$ tons per sq. inch tension, $\cdot 3 \times 5 = 1 \cdot 5$ tons each rivet $= 3360$ lbs., or one rivet at each end would carry nearly double the required load. Notwithstanding this we cannot have less than two rivets in the tee iron, and should not make them less in diameter than $\frac{9}{16}$ inch to go through $\frac{1}{2}$ inch thicknesses. If these two rivets are placed opposite each other, they will bring the weight upon one side only of the bottom flange, to equalise the loading therefore, we may put them zigzag, as in Figs. 1 and 2, and have in each end,

say . . . . . $2 - \frac{9}{16}$ inch rivets.

PITCH OF RIVETS IN GIRDER FLANGES.

In top flange the extra plate being only $\frac{1}{2}$ inch thick, the riveting must be not more than

say . . . . . 3 inches pitch,

but in the bottom flange it may be increased to 6 inches. It will be better, however, to divide the space between the cross beams uniformly, $\frac{1 \cdot 875 \times 12}{4} = 5 \cdot 625$,

say . . . . . $5\frac{5}{8}$ inches pitch,

except at each cross beam where the rivets will go through the flange on each side of the web, each say, 1 inch from centre line of cross beam on opposite sides.

DEFLECTION.

$$\frac{\cdot 015\, s^2}{d''} = \frac{\cdot 015 \times 30^2}{45} = \cdot 3 \quad . \; . \; . \; . \; . \; \tfrac{3}{10} \text{ inch.}$$

CAMBER.

Say . . . . . 1 inch.

EXPANSION.

$$\frac{30 \times \frac{7}{16}}{100} = \cdot 131, \text{ say} \quad . \; . \; . \; . \; . \; \tfrac{1}{8} \text{ inch.}$$

STIFFENERS.

Two outriggers, as shown in Fig. 129. These cannot be calculated. Make them of tee iron, say $4 \times 3 \times \frac{3}{8}$, and let cross beams project 15 inches to take them. Two rivets in top and two in bottom.

LENGTH OF BEARING SURFACE.

Assume York stone template, and stock brickwork, carrying safely 12 and 4 tons per sq. foot respectively.

Total load on one girder . . 13·125 tons.
Width of bottom flange . . 6 inches.

$$l = \frac{W}{s\, b} = \frac{13 \cdot 125}{12 \times \cdot 5} = 2 \cdot 1875 \text{ feet,}$$

$\frac{2 \cdot 1875}{2} = 1 \cdot 09$ feet; or, say, 13 inches, but it would be more convenient to widen the end by riveting a bearing plate on the bottom flange, say, 9″ × 9″ × ½″; then the width being 9 inches instead of 6 inches, the length would be $\frac{13 \cdot 125}{12 \times \cdot 75}$ $= 1 \cdot 46$, $\frac{1 \cdot 46}{2} = \cdot 73$.

Say . . . . . 9 inches.

CHAMFER ON STONE.

$$\frac{W + \sqrt{\text{span}}}{10} = \frac{13 \cdot 125 + \sqrt{30}}{10} = 1 \cdot 86.$$

Say . . . . . 2 inches.

AREA OF STONE.

Bearing surface of girder 9 × 9 = 81 sq. inches.

$\frac{81 \times 12 \text{ tons on stone}}{4 \text{ tons on brick}}$ = . . . 243 sq. inches.

WIDTH OF STONE.

2-inch chamfer + 9-inch girder + 2½-inch margin at back = ... 13½ inches.

LENGTH OF STONE.

$\frac{243 \text{ sq. inches area}}{13 \cdot 5 \text{ inches width}}$ = ... 18 inches.

THICKNESS OF STONE.

½ width of girder to ½ length of stone ∴ 3 inches to 9 inches, say . . . . . 6 inches.

Each stone will then be 18″ × 13½″ × 6″ unless made continuous, when they would be 8′ × 13½″ × 9″, the thickness being increased to prevent breaking across the middle by settlement of brickwork.

The bearing-plates should be let flush into the stone, and the end floor plates will then have a level bed to rest upon.

TOTAL LENGTH OF GIRDERS.

30 feet span + (9 inches length bearing surface of girder × 2) = ... 31 feet 6 inches.

JUNCTIONS OF BARS.

At the junctions of the diagonal bars it will be necessary to put in a distance piece or washer equal in thickness to the webs of the top and bottom T irons, viz. ½ inch thick, and of a diameter not less than the width of the bars. If of wrought iron they may be round or square, as shown at Figs. 144 to 147, with a rivet through the centre. They may also be ornamented by placing a cast-iron boss, say 6 inches diameter, on the outside, as shown in Fig. 143, and using a small round washer between the bars, but the connection must then be made by a bolt and nut instead of a rivet, and the angle-iron struts must be placed on the inside of each girder, leaving only the flat ties on the outside.

WEIGHT OF BRIDGE COMPLETE.

Main girders :—

		Lbs.
Top flange . . .	$6 \times 4 \times \frac{1}{2}$ ⊤ iron	
	$= 4{\cdot}77$ area $\times 3\frac{1}{3} \times 31{\cdot}5$ . . . . =	500
Extra plate . .	$6 \times \frac{1}{2} \times 3\frac{1}{3} \times 16$ feet =	160
		660
Add same for bottom flange		660
Add extra length of plate	$6 \times \frac{1}{2} \times 3\frac{1}{3} \times 7{\cdot}5$ =	75
Tension bars .	$5{\cdot}5 \times 3\frac{1}{4} \times \frac{3}{8} \times 3\frac{1}{3} \times 2$ =	45
Do. .	$5{\cdot}5 \times 2\frac{3}{4} \times \frac{3}{8} \times 3\frac{1}{3} \times 2$ =	38
Do. .	$5{\cdot}5 \times 2 \times \frac{3}{8} \times 3\frac{1}{3} \times 2$ =	28
Do. .	$5{\cdot}5 \times 1\frac{1}{2} \times \frac{3}{8} \times 3\frac{1}{3} \times 2$ =	21
Compression bars .	$5{\cdot}5 \times 1{\cdot}93 \times 3\frac{1}{3} \times 2$ =	70
Do. .	$5{\cdot}5 \times 1{\cdot}32 \times 3\frac{1}{3} \times 2$ =	46
Do. .	$5{\cdot}5 \times {\cdot}81 \times 3\frac{1}{3} \times 2$ =	30
Do. .	say do. =	30
End bars . .	$3{\cdot}75 \times 1{\cdot}93 \times 3\frac{1}{3} \times 4$ =	96
Do. web plate	$1{\cdot}375 \times 3{\cdot}83 \times \frac{1}{4} \times 40 \times 2$ =	106
Bearing plates .	${\cdot}75 \times {\cdot}75 \times \frac{1}{2} \times 40 \times 2$ =	22
Washers 8/ 3″ diam. $\times \frac{1}{2}$″ thick, say . .	=	8
Stiffeners $4 \times 3 \times \frac{3}{8}$ ⊤ iron		
$= 2{\cdot}5 \times 3\frac{1}{3} \times 5 \times 2$	=	84
Projection of beams to receive stiffeners		
$4{\cdot}02 \times 3\frac{1}{3} \times 1{\cdot}25 \times 2$	=	33
		2052
Add same for other girder	=	2052
Cross beams $17 \times 7 \times 4{\cdot}02 \times 3\frac{1}{3}$ . .	=	1595
	Forward .	5699

	Lbs.
Brought forward .	5699
Packing pieces $2 \times 7 \times 4 \times \frac{1}{4} \times 3\frac{1}{3}$ . . =	15
Floor plates $(30 + 3{\cdot}75) \times 6 \times 12$ lbs. per foot . . . . . =	2430
	8144
Add $2\frac{1}{2}$ per cent. for rivet heads . . . =	204
Total weight wrought iron (= 3·67 tons) . =	8348
Ornamental cast-iron bosses 16 at, say, $3\frac{1}{2}$ lbs. each . . . . . =	56
Total weight of whole bridge (= 3·7 tons) . =	8404

The estimated weight of structure was taken at 3·75 tons and the finished design may therefore be proceeded with.

ESTIMATE OF COST.

The cost will be variable according to the market values of material and labour, but would be made up in the following form :—

	£	s.	d.
Wrought iron at £10 per ton . .	36	3	10
Cast-iron ornament at 15*s.* per cwt. .	0	7	6
Labour at £3 10*s.* per ton . .	12	15	3
Erection at £1 10*s.* „ . .	5	9	8
	£54	16	3

PAINTING.

	£	s.	d.
Priming coat of red lead paint, and scraping and painting three coats best oil colour, finished approved tint, approximately 100 yards superficial, at 1*s.* 3*d.* . .	6	5	0
Extra for gilding on ornamental bosses, say	1	0	0

TOTAL COST OF EACH BRIDGE.

	£	s.	d.
Ironwork and labour . . .	54	16	3
Painting	7	5	0
	£62	1	3

Or say, £62.

Exclusive of brickwork and masonry, and of any charge for testing.

FINISHED DRAWINGS.

The design will have been sketched out now in its various parts, and the finished or "contract" drawings may be proceeded with. See Plate 12.

WROUGHT IRON ROOI

Fig. 152

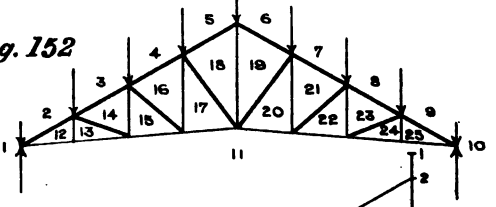


Fig. 153

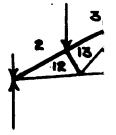


Fig. 158

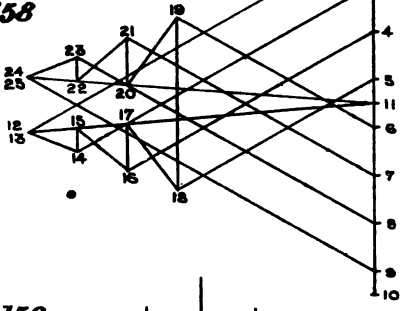


Fig. 159

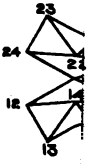


Fig. 156

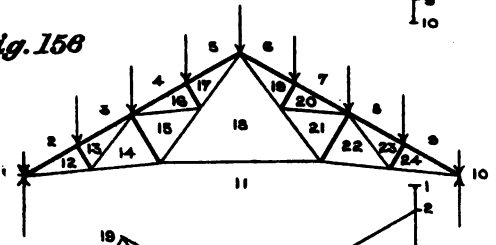


Fig. 157

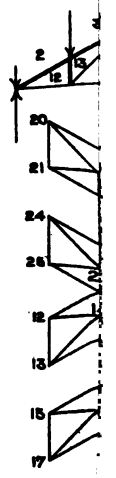
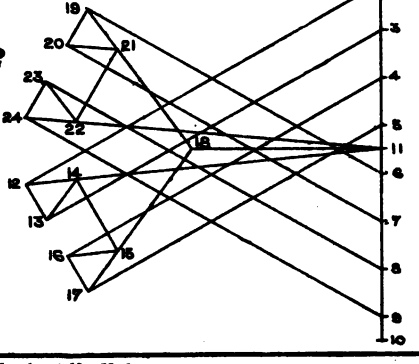
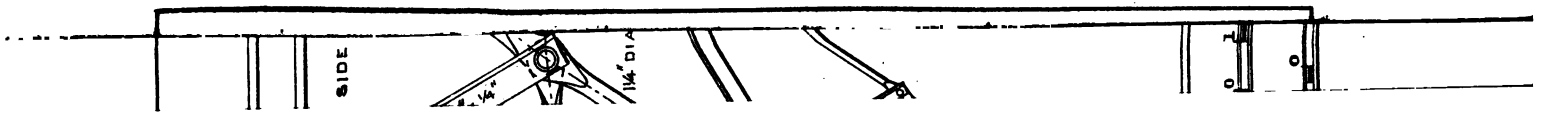


Fig. 162



E. & F. N. Spon, London & New York.


SIDE
1¼"
1¼" DIA

CHAPTER XI.

WROUGHT-IRON TRUSS, FOR ROOF OF 45 FEET SPAN, WITH ANGLE IRON PURLINS, FIR COMMON RAFTERS, SLATE BATTENS, AND COUNTESS SLATES.

(See Plates 13 and 14.)

A ROOF truss is an assemblage of parts so arranged as to form a series of triangles; they may be of wood, of wood and iron combined, or of iron only.

"A hipped roof is more expensive than one with gable ends, but the hipped end is a considerable support to the roof, and itself offers much less resistance to the wind than a gable."—*Maynard.*

NUMBER OF BAYS IN TRUSS FOR ORDINARY PITCH.

The number of bays is regulated, among other things, by the span suitable for common rafters of ordinary section, so that each purlin may rest upon a junction of a strut with principal rafter.

1 bay in rafter is suitable for roofs up to					15	feet	span.
2 bays in rafter are		„	„	15	„ 30		„
3	„	„	„	„	25	„ 50	„
4	„	„	„	„	40	„ 75	„
5	„	„	„	„	60	„ 100	„

For this roof with 45 feet span, say 4 bays.

PITCH OR RISE OF ROOF.

The proper rise for a slated roof in England is considered to be $\frac{1}{4}$ the span = 26° 33′ 54″, say 26½°, but 30° is more likely to keep weather-tight, only costs a little more, and is easier to set out.

For railway stations and such like places, where an occasional leak is of little importance, a pitch of $\frac{1}{5}$ span is sufficient.

In this case . . . . . say 30°.

CAMBER OF TIE ROD.

Say 1 inch per foot run of sloping tie, or a total camber measured in centre of $\frac{1}{30}$ to $\frac{1}{40}$ span. The camber assists to give an appearance of strength and lightness, but actually increases all the stresses by reducing the effective depth of the truss.

OUTLINE OF TRUSS.

Figs. 152 to 157 show various arrangements of trussing suitable for straight rafter roofs, from 40 to 75 feet span, each rafter being divided into four bays.

Fig. 152 has the tie rod divided into eight equal bays by means of vertical ties from the rafters.

Fig. 153 has the struts placed at right angles to the rafters, in order to minimise their length, struts being the most expensive members of the truss. This gives seven

unequal divisions of the tie rod, and with the pitch and camber fixed upon, the centre bay would be unsightly.

Fig. 154 shows a slight modification of Fig. 153, to equalise the divisions of the tie rod.

Fig. 155 shows only six equal divisions of the tie rod; this would no doubt give sufficient support to it, but the general appearance is not so good as that of Fig. 154.

Fig. 156 is upon the principle of the trussed rafter, with secondary trussing; it is known in America as "Fink" trussing. It is economical as regards material, but the joints are a little more expensive.

Fig. 157 was adopted by J. W. Barry, for a span of 63 feet, at Ealing Broadway Station, because "the load (of snow and dead weight) acts vertically, therefore the struts should be placed vertical."

SELECTION OF TYPE.

In making a selection for the present case, we have three points to consider, viz. appearance, weight and cost. The latter is the most important, and, as will be shown presently, it depends primarily upon the stresses in the individual parts. We must therefore construct a stress diagram for each type.

STRESS DIAGRAMS.

In comparing these we will assume the load to be unity over the whole truss, and acting vertically. The side pressure

of the wind need only be considered in a queen-post truss, a shallow arched truss, or a roof of very large span.

For the method of constructing stress diagrams in general see the author's 'Strains in Ironwork.'

DISTRIBUTION OF LOADING.

The common method is to assume the load equally distributed, and that each point of support carries half the load between itself and the next support, so that a truss with the principal rafters divided into four bays would have the loads $= \frac{1}{16}, \frac{1}{8}, \frac{1}{8}, \frac{1}{8}, \frac{1}{16}$ on each = unity total load. But assuming the common rafters to be in the condition of continuous beams, Rankine shows that the distribution of loading would be $\frac{11}{224}, \frac{32}{224}, \frac{26}{224}, \frac{32}{224}, \frac{11}{224}$ on each principal rafter = unity total load.

See also "Theorem of Three Moments," Molesworth, p. 164.

The common method is sufficiently accurate for all practical purposes, and the stress diagrams may be proceeded with.

STRESS DIAGRAM FOR TRUSSED RAFTER ROOF.

In the case of Fig. 156, there is a difficulty in drawing the stress diagram by the reciprocal method alone, because when space 15 is reached there are three unknown parts dependent on each other. Rankine's "Method of Sections" or the "Principle of Moments" may be used to determine the stress in the tie rod, and its value may be transferred to the diagram; thus,

Moment at centre, with unity total load, due to

load 4—5 × hor. dist. from centre = $\frac{1}{8}$ × 5·63 = 0·70375
„ 3—4 × „ „ = $\frac{1}{8}$ × 11·26 = 1·40750
„ 2—3 × „ „ = $\frac{1}{8}$ × 16·89 = 2·11125
„ 1—2 × „ „ = $\frac{1}{16}$ × 22·50 = 1·40625

5·62875

Moment at centre due to reaction of load on whole rafter = $\frac{1}{2}$ × 22·5 = 11·25000

Less . 5·62875

Difference = 5·62125

Then stress in tie-rod =
5·62125 ÷ 11·725 depth of truss = . . . ·48

Another and simpler method of finding the stress on 11-18 is to assume the subdivisions 12 to 17 absent, then the truss and loading will be as Fig. 164, and the diagram as Fig. 165, from which the stress on the centre tie may be transferred to the original diagram, Fig. 162.

It will also be observed that 16-17, Fig. 162, is in line with 12-13; if, then, when line 14-15 is reached, 4-16 and 5-17 be drawn indefinitely and cut off at 16-17, then 14-15 and 16-15 may be drawn, also 17-18 and 11-18; this follows because 14-15 is parallel to 12-13 in Fig. 156, and the stress on 15-16 is equal to that on 13-14. This, however, is not so scientific a method of procedure as the others.

Still another method of obtaining the stress diagram is shown by a correspondent in *Engineering* for 29th Dec. 1893, p. 798.

COMPARATIVE ECONOMY OF TRUSSING.

Mr. Thomas Gillott* compares the value of different types by taking the products of the lengths of the various members by the stress upon them, dividing the sum of those in tension by 4 tons per sq. inch, and the sum of those in compression by $1\frac{1}{4}$ tons per sq. inch. The results added together for each truss give their comparative weights.

WEIGHT OF IRON ROOF TRUSSES.

The comparative weights found above for unity load × actual load on each truss in tons × 2 to 3·5 constant = approximate actual weight in lbs., but for ordinary spans the weight of truss is seldom taken out very carefully, as it bears so small a proportion to the whole load.

Hurst, 13th edition, p. 70, gives the following table of weight of iron trusses :—

20 feet span	.	.	=	270 lbs.
30 „	.	.	=	620 „
40 „	.	.	=	1135 „
50 „	.	.	=	1890 „
60 „	.	.	=	3000 „

In Merriman's 'Roofs and Bridges' the weight of truss in lbs. is given as

Timber . . . = $\frac{1}{2} a l (1 + \frac{1}{10} l)$.

Iron . . . = $\frac{3}{4} a l (1 + \frac{1}{10} l)$.

Where l = span feet, a = truss spacing feet.

Another approximate rule, near enough for ordinary purposes, is

·75 (span feet^2) = weight of truss in lbs.

* Trans. Soc. Eng., 1881.

SUMMATION OF STRESSES.

In order to save space, we will here show only one example in detail, the others would be similar, and should be worked out for practice.

Left.	Right.	Stress.	Length.	Product.
2—12	9—24	1·025*	6·5	6·6625
3—13	8—23	·965*	6·5	6·6625
4—16	7—20	·9 *	6·5	6·6625
5—17	6—19	·84 *	6·5	6·6625
12—13	23—24	·11	3·0	·33
14—15	21—22	·215	6·0	1·29
16—17	19—20	·11	3·0	·33
				28·6000
				2
			Total +	57·2000

Left.	Right.	Stress.	Length.	Product.
11—12	11—24	·89	7·2	6·408
11—14	11—22	·765	7·2	5·508
13—14	22—23	·13	7·2	·936
15—16	20—21	·13	7·2	·936
15—18	18—21	·305	7·2	2·196
17—18	18—19	·43	7·2	3·096
				19·080
				2
				38·160
	11—18	·48	16·6	7·968
			Total −	46·128

* As the section of principal rafter must be uniform, the maximum stress of 1·025 must be taken for each bay.

Sum of tension members	46·128 ÷ 4	=	11·532
„ compression „	57·2 ÷ 1¼	=	45·760
Comparative weight .		=	57·292

The comparison of the six trusses will be as follows :—

Truss Fig.	1	=	60·462
„	2	=	59·809
„	3	=	58·830
„	4	=	59·318
„	5	=	57·292
„	6	=	58·583

In general the cost will vary nearly as the weight, we may therefore decide that Fig. 156 is the type to adopt for our roof.

This type has been adopted for roofs up to 90 feet span, but Prof. Unwin recommends that it should not be used for spans exceeding 60 feet.

NATURE OF COVERING.

The cheapest covering is galvanised corrugated iron upon curved trusses ; a series of such roofs of 45 feet span designed by the author and fixed 60 feet above ground-level cost £8 per square measured on plan, *complete, including all charges.* A plain curved roof is always unsightly, a raised skylight running throughout the length of it in the centre improves the appearance, but adds to the cost.

A straight rafter roof is more suitable for slating or glazing.

Iron trusses are generally exposed to view on under side.

When covered with slates, slate battens $3\frac{1}{2} \times 1$, or $3 \times 1\frac{1}{4}$, may be used for laying them on, but if the interior be exposed to gusts of wind, boarding must be used to prevent the slates from blowing up, and when the interior has to be kept draught-tight the boarding must be covered with roofing felt. In exposed situations steeper pitch or heavier slates must be used, iron roofs are often too flat, $\frac{1}{5}$ span being a common proportion for the rise.

LOAD ON ROOFS.

Wray's 'Theory of Construction,' p. 195, gives a very complete table. All we require for the present roof are the following:—

Countess slating . . .	8	lbs.	per sq. foot.
Slate battens for ditto . .	$1\frac{1}{4}$	„	„
Slate boarding or sarking ($\frac{3}{4}$″)	$2\frac{1}{2}$	„	„
Common rafters . . .	3	„	„
Snow	5	„	„
Wind (average vertical load on straight rafter roofs) .	30	„	„
Purlins and trusses (iron) .	7	„	„

Amounting to say, 56 lbs. per foot superficial, measured on the sloping surface.

The late architect on a large railway always calculated his roofs for a vertical load of 60 lbs. per foot superficial, to include all contingencies, which is apparently a fair approximation to actual conditions, as excessive corrosion must be provided against. It must be borne in mind, however, that some roof trusses, especially when curved, require to be con-

sidered when loaded unsymmetrically, as by the wind acting on one side only.

The actual effect of the wind pressure on this roof truss is discussed in a paper by the author, entitled "Wind Pressure on Roofs."* It is shown that no material error arises in this case through assuming a vertical wind load.

DISTANCE APART OF TRUSSES

varies from 5 feet in small roofs to 30 feet in large roofs, and may be about $\frac{1}{4}$ of span in roofs of moderate size. A common distance is 10 feet to 12 feet 6 inches.

The most economical distance depends upon the stress in the compression members, compared with their length. By placing the trusses further apart, the sections of the compression bars will be increased, while their length remains the same, and therefore a greater thrust per sq. inch of section may be put upon them.

From existing examples a suitable formula would appear to be

$$\text{Distance apart} = \frac{\text{span}}{\cdot 03 \text{ span} + 2}.$$

According to Matheson, the distance apart should be $\frac{1}{8}$ to $\frac{1}{4}$ the span: "if these limits be overstepped there will be an unprofitable employment of material."

$\frac{45}{4}$ = say . . . . . 11 feet 3 inches.

The same type may be used for larger or smaller spans, by varying the distance apart of the trusses, thus—

$$\frac{\text{Span} \times \text{distance apart}}{\text{proposed span}} = \text{required distance apart.}$$

* Spon, 2nd edition, 1893, 6*d*.

TOTAL LOAD ON EACH TRUSS.

We are now in a position to determine this.

For length of rafter, $l = \sqrt{(\frac{1}{2} \text{ span})^2 + \text{total rise}^2}$, or when pitch = 30°, then length of rafter = twice total rise of roof; or $\frac{1}{2}$ span × 2 tan 30° = 22·5 × 1·155 = 25·9875 say 26 feet, which may also be scaled off from Fig. 156.

Area of roof surface supported by truss = twice length of rafter × centre distance of trusses = 2 × 26 × 11·25 = 585 feet sup.

Total load = 585 feet × 60 lbs. = . . 35,100 lbs.

ACTUAL STRESSES ON TRUSS.

The actual stresses on the selected truss may now be obtained by multiplying the unit stress by 35,100 for the result in lbs., or by $\frac{35{,}100}{2240}$ = say 15½ for the result in tons.

This will give us the stresses marked on Fig. 15, and the detailed designing of the truss may be proceeded with.

A rough method in designing iron roofs when they were first adopted, was to provide sufficient material for the parts in tension to be strained to 5 tons per sq. inch, and those in compression to 2½ tons per sq. inch, irrespective of their length, but this is a very unsafe and objectionable method.

PRINCIPAL RAFTER.

In small roofs generally ⊤ iron, but occasionally two angle irons back to back, or two channel bars may be used, with ends of bracing bars between them. When loaded

between the points of support,* the top table of ⊤ iron is made as small as will resist the thrust without buckling, and the vertical web deep to resist the cross strain, sufficient dimensions being given to the parts to allow of proper connections with the other members of the truss. When the load is applied over the points of support only, the breadth and depth are generally made equal.

The section must be designed for the maximum stress, and being uniform, will involve a slight waste of material, which is avoided in larger structures.

The greatest thrust is at the foot of the rafter, and in this design, as the purlins come upon the junctions, there is no bending stress beyond that due to its own weight.

Maximum stress . . .	15·89 tons, Fig. 166.
Unsupported length . .	6·5 feet.

In roofs the principal rafter is so favourably situated as regards position and fixing, that Box considers that a ⊤ iron may be safely strained to 5 tons per sq. inch.

$$= 4 \times 4 \times \tfrac{7}{16} \text{ ⊤ iron.}$$

If taken simply as a pillar fixed one end, Box † would give the necessary size as

$$= 4\tfrac{1}{2} \times 4\tfrac{1}{2} \times \tfrac{1}{2} \text{ ⊤ iron.}$$

Gordon's formula ‡

$$\text{B. W. tons} = \frac{16\,S}{1 + \frac{l^2}{ad}}$$

* An excessive strain is caused by the combination of a transverse and direct stress, which should always be avoided if possible. With a little care the trussing can generally be arranged so that a strut comes under each purlin, but there are some cases where the load is necessarily applied at shorter intervals.

† 'Strength of Materials,' p. 290.

‡ Hurst's 'Handbook,' 13th edition, p. 43.

where S = sectional area sq. inches, l = length in inches, a = constant, d = least diameter or breadth in inches.

Taking a = 1,500 for cross, angle, or tee ; factor of safety = $\frac{1}{4}$, and $\frac{a}{2}$ for only one end fixed, would give safe load of nearly 3 tons per sq. inch, or say = $4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{5}{8}''$ tee iron.

Our rule on p. 150, for lattice girder struts, would show at least a $6'' \times 6'' \times \frac{3}{4}''$ tee iron, but in that case vibration had to be allowed for.

We shall therefore be quite safe in taking the required size as . . . . . $4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{5}{8}''$

The joints must be left for consideration until the cross sections of all the members are decided upon.

STRUTS.

These may be wrought or cast iron, but are generally wrought, unless an ornamental effect be desired by their use. They must be of a section suited to resist thrust, such as angle or tee iron, or two flat bars with distance pieces between, or channel bars with distance pieces. The distance pieces may be of gas pipe or of cast iron.

COMPARATIVE STRENGTH OF STRUTS ACCORDING TO MODE OF FIXING.

"If a pillar or strut of given length, pivoted at ends, be taken as unity, the same formula will give strength of pillar with fixed ends by taking it as of half the length ; and one end fixed and one pivoted, by taking two-thirds the length." —*Max am Ende.*

"Mr. Davies, of the Crumlin Works, found that when

angle iron or other struts were reduced to a flat plate at the ends, for the purpose of riveting, the strength was reduced by from one-tenth to one-third.—*Unwin.*

Ordinary struts are all more or less fixed at ends and a rough approximate rule for safe load in tons per sq. inch sectional area in wrought iron, is

$\frac{\text{Length}}{\text{Least width}}$	Tons per sq. in.
60	$\frac{1}{2}$
50	1
40	$1\frac{1}{2}$
30	2
20	3
10	4

This table is for dead loads, as in roofs; take half for live loads, as in bridges and cranes.

SIZE OF STRUTS.

Let the short struts be made of angle iron.

Stress = 1·71 tons, Fig. 166.
Unsupported length = 3 feet,

taking Gordon's formula as before, but with a further modification to $\frac{a}{4}$ to allow for both ends pivoted.

$$\text{Safe load per sq. inch} = \frac{4}{1 + \frac{l^2}{375\,d^2}},$$

and taking 2 inches as the least dimension, we get a safe load of 2·14 tons per sq. inch, which will require an angle iron, say . . . . . 2″ × 2″ × $\frac{1}{4}$″

Let the long struts be made of two bars with distance pieces between. They will be in the condition of pillars liable to bend on the least diameter, which will be the *width* of the bars.

Stress = 3·33 tons, Fig. 166.
Unsupported length = 6 feet,

same formula as last and 2 inches for least dimension gives a safe load of ·9 tons per sq. inch, or 3·7 sq. inch, which would be rather much for two bars.

Say $2\frac{1}{2}$ inches for least dimension, then we have safe load 1·24 tons per sq. inch, or 2·7 sq. inch section,

and $$\frac{2\cdot7}{2 \times 2\frac{1}{2}} = 0\cdot54 \text{ inch thick.}$$

say 2 bars, each . . . . . $2\frac{1}{2}'' \times \frac{9}{16}''$.

Long inclined struts tend to deflect by their own weight, and although allowance is not usually made for this, the fact must not be lost sight of in designing large roofs.

The space between the distance pieces should be such that the ratio of width of bar to length of strut is the same as thickness of bar to space between distance pieces,

$2\frac{1}{2}''$: 6′ 0″ :: $\frac{9}{16}''$: 1·35 feet or less.

∴ the 6 feet must be divided into 5 spaces of $\frac{6}{5}$ = 1·2 feet each.

TIE RODS OR TENSION BARS.

Generally a single round bar is used; occasionally, where a greater stress is to be resisted, one or more flat bars with the largest dimension vertical; with flat bars the joints are sometimes simpler. With tension members the length is of

no consequence unless the piece is horizontal or nearly so, and then it only needs consideration in extreme cases.

The working strength of wrought iron in tension is 5 tons per sq. inch, but in the tie rods of a roof there are various welds, at each of which a loss of 20 per cent. of the original strength may occur, hence it will not be proper to estimate more than 4 tons per sq. inch safe load, unless increased diameter is given at the welds.

Taking round rods at 4 tons per sq. inch, the following will be the dimensions:—

Inner ties . .	stress	2·02	tons =	$1\frac{3}{16}$"	diameter.
Middle ties .	"	4·73	"	$1\frac{1}{4}$	"
	"	6·67	"	$1\frac{7}{16}$	"
Main ties . .	"	7·44	"	$1\frac{9}{16}$	"
	"	11·89	"	$1\frac{15}{16}$	"
	"	13·80	"	$2\frac{1}{8}$	"

KING ROD.

If thought necessary a light rod may be put in to prevent the tie rod from sagging, similar to that shown in Fig. 157, but it is hardly wanted in this case.

SHOES OR CHAIRS.

Each end of the truss must rest on a shoe or chair to allow of proper bedding on the supports. Generally this is made to form part of the connection between tie rod and foot of principal rafter. It may be cast or wrought iron.

JOINTS AND CONNECTIONS.

These may now be designed, and should be drawn out to a large scale, showing all details. See Figs. 171 to 179, and complete truss, Fig. 180.

The diameter of pin or bolt through end of tie should not be less than diameter of tie, and sectional area through sides of eye should not be less than $1\frac{1}{4}$ times sectional area of pin. In roofs exceeding 45 feet span means should be provided for tightening up the parts, generally by cotters through ends of main tie rod or coupling screw with right and left hand thread in centre of rod; they are sometimes put in roofs exceeding 20 feet span.

"In fixing iron roofs, care should be taken to have the connections properly forged, and the bolts and keys of sufficient strength."—*Hurst.*

In clothing the outline of truss the axes of centres of gravity of the members should coincide as closely as possible with the original lines of the "frame diagram," in order that there may be no distorting elements when the truss is loaded. See stroke and dot lines in Figs. 171 to 179.

FIXING OF TRUSS.

Iron trusses, unless arched, should only be fixed at one end on account of change of temperature, causing expansion and contraction. The fixed end should be the side from which the strongest winds blow, say the south-west in the neighbourhood of London, unless more exposed to any other quarter.

The chair at fixed end should be bolted down by lewis bolts to a sufficiently massive stone bonded in the wall, or be

fixed by cleats or joggles. The chair at other end should rest on a flat plate of cast iron, bolted down or otherwise secured, and in larger roofs friction rollers should be interposed.

WIND TIES.

Light wrought-iron rods, to prevent the straining of a roof by an oblique wind, are frequently fixed between the trusses, passing up and down from bottom of king bolt of one to ridge of next, as in Fig. 167.

Or they are fixed diagonally across say three bays, from the springing to the ridge, as in Fig. 168.

These are not calculated but may be made of a diameter $= \frac{1}{8}$ inch per 10 feet span $+ \frac{1}{4}$ inch, for the former case; and $\frac{3}{8}$ inch per 10 feet span for width, and $\frac{1}{16}$ inch per 10 feet span for thickness, in the latter case, being fixed at the crossing of each truss.

Many engineers only put wind ties in the last two or three bays of a long roof, considering that if the purlins are properly connected the remainder of the structure will be stiff enough; they are usually omitted altogether in roofs that are not very much exposed.

ERECTION OF ROOF.

When roof trusses are put together on the ground and lifted into place by a derrick pole, care should be taken to sling them from two points sufficiently far apart to avoid buckling the tension rods, which might occur when they were lifted from the junction of the principal rafters if there were any surging of the tackle.

For facility of carriage and transport, this truss might be

sent out from the factory in three pieces, viz.:—one main tie rod and two trussed rafters, each with one of the apex plates, temporary bolts and wood blocks being used where necessary, to keep the parts firmly together.

COST OF TRUSSES.

From Mr. Gillott's investigations the respective costs of ties and struts to transmit a unit of stress (1 ton) through a unit of length (1 foot) = 2·35 pence and 3 pence.

The relative cost of trusses may then be found by summing the products of length by actual stress for the various members, and multiplying by the above figures.

Applying this method to our selected truss.

		£	s.	d.
(−) 46·128 × 15·5 × 2·36	=	7	0	7
(+) 57·2 × 15·5 × 3	=	11	1	8
		18	2	3

The complete list including the other types will be,

	£	s.	d.
Fig. 1	18	14	2
„ 2	18	10	11
„ 3	18	7	0
„ 4	18	8	2
„ 5	18	2	3
„ 6	18	16	7

so that the selected type comes out the cheapest as well as the lightest.

COST OF ROOFING COMPLETE.

In Walmisley's paper on Iron Roofs (Soc. Eng. Trans., 1881) various examples are given, the cost of which varied from £6 to £50 per square measured on plan, large roofs costing the most. £10 per square appears to be a fair price for a roof 40 to 50 feet span.

CHAPTER XII.

SPECIFICATION TESTS AND TABLE OF WORKING STRESSES.

SPECIFICATION TESTS OF CAST IRON.

THREE bars, each 3 feet 6 inches long, 2 inches deep, and 1 inch wide, to be cast on edge, in a dry mould, from each melting at which any of the specified work is cast. These bars are to be tested separately as follows:—The lower side or thin edge of the casting to be placed downwards * upon rigid bearings, with 3 feet clear span; each bar to deflect not less than $\frac{3}{10}$ inch with a load of 25 cwt. in centre, having a bearing not more than 1 inch wide upon the bar, and to break with a minimum load of 28 cwt., and an average upon the three bars of not less than 30 cwt.

Samples prepared in the lathe are to bear $2\frac{1}{2}$ tons per square inch tensile strain before loss of elasticity, and to break with not less than 7 tons per square inch, or an average on three samples of $7\frac{1}{2}$ tons.

TESTS OF CAST IRON FOR PIPE-MAKING.

"A bar of metal 40 inches long, 2 inches deep and 1 inch wide, the weight of which must not exceed 21 lbs., shall, when

* Placed the other way up, a reduction of 15 to 20 per cent. in the apparent strength may occur.

supported on edge at points 36 inches apart, sustain a load of 3000 lbs. supported at the middle of its bearing for one hour, and shall under this load deflect at least $\frac{3}{8}$ inch at the middle, or a bar 8 inches long and 1 inch square in section shall sustain a load of 8 tons tensile stress for one hour."

SPECIFICATION TESTS OF COMMON WROUGHT IRON.

Class.	Tons per square inch, Tensile Strength.	Contraction per cent. at point of Fracture.
Rivet iron	22	20
Rods, bars and angles ..	21	12½
Plates	20	10

SPECIFICATION TESTS OF WROUGHT IRON

(Bridge and Girder Work).

Class.	Tons per square inch, Tensile Strength.	Elongation * per cent. at 20 tons.	Contraction per cent. at Point of Fracture.
Rivet iron	25	10	30
Rod and bar iron.. ..	24	7½	20
Angle and tee iron ..	22	6	15
Plates, with grain.. ..	21	4½	10
Plates, across grain ..	18	2½	5

* In a length of 8 inches.

SPECIFICATION TESTS OF WROUGHT IRON AND STEEL.

(Shipbuilding.)

Class.	Tons per square inch, Tensile Strength.	Elongation * per cent on Fracture.	Toughness.†
Rivet iron	26	25	650
Rod and bar iron	24	15	360
Angle and tee iron	22	12½	275
Iron plates, with grain ..	20	7½	150
Iron plates, across grain ..	19	6	114
Steel plates (both directions)	28	20	560
Steel bars and angles ..	30	25	750

* In a length of 6¼ inches.

† Should the actual elongation in sixteenths of an inch, multiplied by the stress in tons per square inch, upon rupture, be more than 10 per cent. under the amounts given in the last column, the material will be rejected.

WROUGHT IRON.—Cold bending in vice—½ inch plate 35°, ⅜ inch plate 55°, 5/16 inch plate 63°, ¼ inch plate 70°, rivet iron to double close, without cracking.

STEEL.—Steel plates should be capable of being bent to an inside radius of 1½ times their thickness, when heated to a low cherry red, and cooled in water of a temperature of 28° C. = 82° F.

For Admiralty tests see 'Molesworth,' p. 28 (21st edition).

Lloyd's regulations allow a reduction of 20 per cent. in the scantlings of a steel ship as compared with iron, but the total weight of material used is only about 14 per cent. less. The cost is about the same in steel or iron.

TABLE OF WORKING STRESSES.

The common working stresses in tons per square inch that may safely be allowed in ordinary structures are as follows —

	Tension.	Compression.	Shearing.	Bearing.
Cast iron	1½	7	2½	8
Wrought iron ..	5	4	4	5 to 7½
Mild steel	7½	6	5	10

	Single Shear.	Double Shear.
Rivets, iron ..	5	7½
Rivets, steel ..	6	9

	Across Grain.	With Grain.
Oak	350 lbs. = $\frac{3}{20}$ tons	675 lbs. = $\frac{3}{10}$ tons
Fir	250 „ = $\frac{1}{10}$ „	450 „ = $\frac{2}{10}$ „

Parts wholly in compression and exceeding five diameters in length should be calculated as struts or pillars. This applies also to girder webs, but a simple rule, to avoid calculation as a pillar, is to allow 3 tons per square inch shear stress up to 2 feet deep, and 2 tons when deeper.

For safe load on materials see Chapter IV. p. 43.

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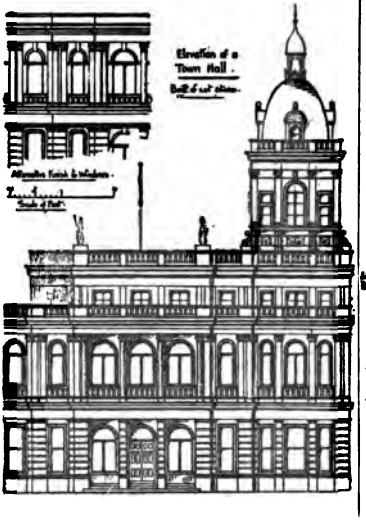

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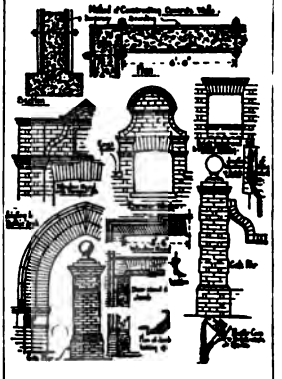

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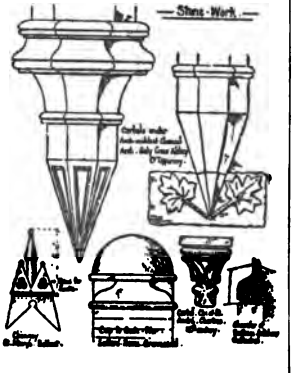

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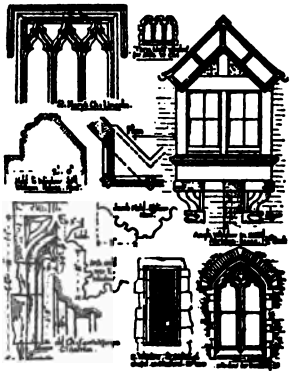

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